BEHAVIOUR OF PRECAST BEAM-TO-COLUMN CONNECTIONS BY USING PARTLY HIDDEN CORBEL

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ABSTRACT

The performance of precast concrete structures is greatly influenced by the behaviour of beam-to-column connections. A single connection may be required to transfer several loads simultaneously so each one of those loads must be considered in the design. A good connection combines practicality and economy which requires an understanding of several factors; strength, serviceability, erection and economics.

This research focuses on the performance aspect of a specific type of beam-to-column connection using partly hidden corbel in precast concrete structures. The study presents the experimental assessment of the beam-to-column connection in precast concrete frames. The main objective of this study is to investigate the behaviour and capacity (such as the moment resistance) and the failure mechanisms within connection zone of a proposed connection by conducting experimental tests.

Full-scale testing is conducted on three precast concrete exterior beam-to-column connections to determine their suitability as semi-rigid and partial-strength connections. The behaviour of a specific type of beam-to-column connection is examined; the connection uses a partly hidden corbel with different anchorage arrangements in precast concrete structures. The capacity (e.g., moment resistance) and failure mechanism of the connection are investigated, and experimental assessment of the proposed beam-to-column connection is performed under static and reversible loads. The classification and behaviour of the connection are defined by using moment–rotation (M–ϕ) data together with the characteristic response of the beam, which is known as the "beam line," and the fixity factor under certain loadings in a flexurally cracked state. Based on experimental results, beam-line reveals beam-end's moment of resistance, M_E, rotational stiffness, and fixity factor. Besides that, the proposed precast beam-to-column connection BHC2-T2 and BHC3-T2 can be classified as semi-rigid and it falls under zone III (semi-rigid with

medium strength), meanwhile, BHC1-T1 connection falls under zone II (semi-rigid with low strength). This demonstrates its ability to be applied for semi-rigid analysis of precast frames.

ABSTRAK

Keberkesanan bagi struktur konkrit pratuang adalah sangat bergantung kepada ciri-ciri sambungan rasuk-tiang. Setiap satu jenis sambungan mungkin perlu untuk memindahkan beban secara serentak maka setiap beban tersebut perlu diambilkira dalam kerja merekabentuk. Jenis sambungan yang baik mempunyai gabungan dari segi ekonomi dan juga praktikal dimana ia memerlukan pemahaman yang tinggi dalam beberapa faktor seperti kekuatan, kebolehkerjaan, pemasangan dan ekonomik.

Penyelidikan ini memberi fokus kepada aspek keberkesanan sambungan rasuk-tiang dengan menggunakan jenis sambungan 'partly hidden corbel' dalam struktur konkrit pratuang. Kajian ini membentangkan penilaian eksperimen bagi jenis sambungan rasuk-tiang dalam rangka struktur pratuang. Objektif utama bagi kajian ini adalah untuk menyelidik kelakuan dan kemampuan (seperti momen rintangan) dan mekanisma kegagalan dalam zon sambungan bagi sambungan yang telah dicadangkan melalui pelaksanaan ujian eksperimen.

Pengujian berskala penuh telah dilakukan pada tiga jenis sambungan rasuk-tiang luar konkrit pratuang bagi menentukan kesesuaian sambungan tersebut sebagai sambungan separa-tegar dan kekuatan-separa. Ciri-ciri bagi sambungan rasuk-tiang ini telah dikaji; sambungan ini adalah menggunakan jenis '*hidden corbel*' dengan susunan '*anchorage*' yang berlainan dalam struktur konkrit pratuang. Kapasiti (seperti, rintangan momen) dan mekanisme kegagalan sambungan telah diselidik, dan penilaian uji kaji bagi sambungan rasuk-tiang yang dicadangkan telah dijalankan mengikut kaedah beban statik dan boleh balik. Pengelasan dan ciri-ciri sambungan ini ditafsirkan dengan menggunakan data putaran-momen (M- ϕ) bersama-sama dengan tindak balas ciri rasuk, yang dikenali sebagai "*beam line*", dan faktor ketegaran pada beban tertentu dalam keadaan '*flexurally cracked*'. Berdasarkan keputusan kajian yang menggunakan kaedah '*beam line*' ini,

momen rintangan, M_E, kekakuan putaran, dan faktor ketegaran telah diperolehi. Di samping itu, sambungan rasuk-tiang yang telah dicadangkan iaitu BHC2-T2 dan BHC3-T2 dapat diklasifikasikan sebagai separa-tegar dan berada didalam zon III (separa-tegar dengan kekuatan sederhana), sementara itu, sambungan BHC1-T1 jatuh didalam zon II (separa-tegar dengan kekuatan yang rendah). Ini menunjukkan keupayaann sambungan ini untuk digunakan didalam analisis separa-tegar dalam rangka pratuang.

ACKNOWLEDGEMENTS

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbols

%	:	percentage
$^\circ$	:	degree
kN	:	kilo Newton
kNm	:	kilo Newton metre
m	:	metre
mm	:	millimetre
m.rad	:	milliradian
N/mm^2	:	Newton per millimetre square
A_S	:	area of steel
E_c	:	concrete Young's modulus
E_s	:	steel Young's modulus
T/F_s	:	tensile or shear force in bars or connector
I	:	second moment of area
I_{cr}	:	second moment of area of cracked section
I_u	:	second moment of area of uncracked section
S	:	rotational stiffness
S_E	:	connector secant stiffness at limiting beam rotation
S_{RC}	:	connector secant stiffness at moment M_{RC}
K_s	:	normalised connection stiffness
L	:	span of beam
M	:	applied test bending moment
M_{beam}	:	predicted ultimate moment capacity of beam
M_E	:	beam end moment at limiting beam rotation

M_{ED}	:	design beam end moment at limiting beam rotation
M_R	:	moment resistance of beam
M_{RC}	:	moment of resistance of connector
M_{span}	:	predicted ultimate moment capacity at mid-span of beam
M_U	:	test ultimate moment
P	:	applied beam load
b	:	breadth of section
d	:	effective depth to rebar
f_{cu}	:	compressive strength of concrete in beams and columns
f_{cui}	:	compressive strength of infill concrete in connectors
f_y	:	yield stress of reinforcement
h	:	depth of section
l_e	:	embedment length for tie steel bars
l_p	:	plastic hinge length
$(1/r)_{cr}$	:	curvature of the beam
w	:	uniformly distributed load
x	:	depth to neutral axis in beam
x_c	:	depth to neutral axis in connector
z	:	lever arm in beam/connector
δ	:	deformation
γ	:	fixity factor
γ_m	:	partial safety factor
θ	:	rotation of beam or column
ϕ	:	relative beam-to-column rotation
ϕ_E	:	relative rotation at beam rotation limit

Abbreviations

CIDB : Construction Industry of Development Board

CREAM : Construction Research Institute of Malaysia

IBS : Industrialised Building System

MC : Modular Coordination

MRC : Moment Resisting Connection

OBS : Open Building System

RC : Reinforced Concrete

LIST OF APPENDICES

CHAPTER 1: INTRODUCTION

1.1 Background

Every industrialized building system (IBS) using precast elements has unique connections and joints for assembly of these prefabricated elements into the structure. In general, the success of the IBS depends on both the strength and the effectiveness of the connections, and also the ease and simplicity that can be achieved at the site to speed up the construction process (Trikha and Ali, 2004). However, the weakest link and the most critical part of a precast concrete structure is also the joint or connection (Paul Leong, 2006; Choi *et al.,* 2013), particularly the beam-to-column connections. Therefore, the design of connections in IBS is one of the most important steps as the performance of precast concrete structures is greatly influenced by the flexural properties of beam-to-column connections.

A connection must have the ability to transfer forces between the precast concrete elements to achieve a structural interaction when the system is loaded. A single connection may be required to transfer several loads simultaneously so each one of those loads must be considered in the design. A good connection combines practicality and economy which requires an in-depth understanding of the strength, serviceability, erection and economical aspect of the whole system (PCI, 1988). From a structural point of view, this ability of the connection is an essential property and should fulfill the needs in the ultimate as well as in the serviceability limit states. The important matter is to keep in mind that the flow of forces through connection can influence the performance of overall structure elements. The capability of beam-to-column connection can be determined by moment capacity, moment-rotation response and the failure mechanism within connection zone (Hasan *et al.,* 2011).

Rigid connections are more ductile and can enhance the performance of the global precast structure with respect to the lateral stability. However, the problem with rigid connections is difficult to construct and costly. Therefore, one of the objectives of this research is to develop moment connection that can improve and enable a fast and clean precast concrete construction. Besides, the type of connection which has been chosen can be produced and used by any factory/manufacturer.

1.2   Problem Statement

The conventional method of in-situ concrete construction consists of several problems that have long been associated with dirty site conditions, dangerous workplaces, and requires large numbers of unskilled field labour forces. One of the ways to overcome all these problems is using the precast concrete system such as IBS. The use of IBS assures valuable advantages, inter alia, reduction of unskilled workers, less wastage, less volume of building materials, and environmental impact and better quality control (Trikha and Ali, 2004; Maya *et al.,* 2013; Ha *et al.;* 2014).

The success of the precast building greatly depends on the strength and rigidity of the connections between the precast elements to ensure its resistance to the applied loads. Precast building's efficiency also depends on the production tolerances of the precast elements and its designed and actual dimensions. Therefore, precast concrete connections must have similar capabilities or as close as possible as cast-in-situ concrete connections in term of stiffness, strength and ductility. Moreover, in IBS constructions, moment resisting connections may be used to resist the wind or seismic loading, and also improve the resistance to progressive collapse. Thus, the question is whether these proposed connections can be designed as semi-rigid (moment resistance) connections?

The main problem in this context, that there is currently a lack of experimental data for the ductile connections that can be used in development and improvement of moment

resistance connections (Abd. Rahman *et al.,* 2006). Unlike structural steel work connections, many variables exist in a precast connection coupled with effects of cracking, slippage, and rebar debonding. As such, often experimental work is the only way to validate the theory. Moreover, reliable connection behaviour can only be properly assessed by laboratory testing or proven performance (Loo and Yao, 1995). Therefore, there is a necessity to gain high quality experimental data from actual testing, which should be made available to the industry at large as guidance in designing and assessing the connections. Based on the results obtained, the use of the proposed connections with unbraced frame can be assessed.

In previous researches, there are a number of proposed moment connections. However, many of them do not emphasize on the aesthetical aspect of the connection systems, which is part of a requirement from architects. Therefore, this research focusses on developing an economic moment resistance connection with aesthetical values for the precast concrete system. Besides, in the construction industry, a corbel connection normally is a pinned connection. Thus, the goal is to determine the proposed beam-to-column connections design suitability to be constructed as semi-rigid and partial strength connections.

In addition, there are many types of beam-to-column connections developed and available in the market and most of them are patented by manufacturers. Hence, this study is intended to propose a type of connection which is suitable for an open system that can be used by anybody in the construction industry.

1.3 Research Objectives

The research objectives of this research are as follows:

1. To determine the moment-rotation characteristics (e.g. moment resistance, stiffness) by conducting full-scale precast concrete beam-to-column connections test.

2. To determine the classification of connections by using Monforton's Fixity Factor.

3. To evaluate the crack behaviour and mode of failure of beam-to-column connections.

4. To compare the result between experimental results and theoretical values.

1.4 Research Scope

This research focuses on the performance aspect of beam-to-column connection in precast concrete structures. The experimental tests were conducted at CREAM Laboratory to investigate the connection's behaviour and its performance using the moment-rotation and load-displacement relationships, as well as the mode of failure. The study is limited to the moment resistance of the beam-to-column connection in precast concrete frames.

The analytical models using a theoretical approach to predict semi-rigid behaviour were used to compare the results of experimental testing and theory as a guideline.

The precast beams, corbels and columns for this testing were designed based on BS8110: Part1. The recommended methods of design and detailing of reinforced concrete and prestressed concrete were applied for precast concrete.

1.5 Outline of the Research Approach

The outline of this research approach consists of five main chapters as listed below:

• Chapter 1 presents a brief introduction to the issues that relate to research concern, the problem statement, research objectives, scope of the study and the outline of the research.

• Chapter 2 presents the survey on previous literature and studies and reviewing the available method that had been used.

• Chapter 3 describes the detail steps used in the study and presents the theoretical approach for semi-rigid connection. The experimental testing procedure and method of analysis is also described in this chapter.

• Chapter 4 discusses the results and data analysis obtained from calculation and testing. The findings are compared with theoretical results.

• Chapter 5 summaries and conclude the findings and presents the recommendations for future works.

CHAPTER 2: LITERATURE REVIEW

2.1   Introduction

Recently, precast concrete construction method is being widely applied in our construction industry parallel to the endorsement of IBS Roadmap 2003-2010 by the Government of Malaysia (Abd. Hamid *et al.,* 2008) and followed by IBS Roadmap 2011-2015. It is widely known that the conventional method of in-situ construction consists of several problems that relate with dirty, dangerous, and requires large unskilled field labour forces. Alternatively, to tackle all those issues is using the precast concrete system.

From a survey of the available literature, precast concrete can be defined as concrete which is cast in some location other than its position in the finished structure (Abdul Aziz *et al.,* 2004). In general, precast concrete can be categorized into three basic structural forms which are skeletal frame system, load bearing wall system and cell system. A skeletal frame system is achieved by connecting precast columns and beams together with precast flooring/roofing elements supported by the beams. While, load bearing wall system is solid, sandwich or perforated precast concrete panels that can efficiently carry the vertical loads as well as the horizontal loads. A cell system is the structure consists of a number of precast cell units, which are in-situ connected to build the structure (Trikha and Ali, 2004).

2.2   Industrialised Building Systems (IBS)

An industrialised building systems (IBS) can be defined as a building system which involves the industrialised production of building elements as well as erection and assembly of these elements into a building structure with minimum in-situ construction (Trikha and Ali, 2004). There are a few definitions stated by researchers who studied previously were found through their literature such as an integrated manufacturing and construction process with the well-planned organization for efficient management,

preparation and control over resources used, activities and results supported by the used of highly developed components (Lessing *et al.*, 2005). Esa and Nurudin (1998) defined IBS as a continuum beginning from utilizing craftsmen for every aspect of construction to a system that make use of manufacturing production in order to minimize resource wastage and enhance value and users. Whatever the definitions of IBS are, the key concept of IBS is as follows; a construction technique in which components are manufactured in a controlled environment (on or off site), transported, positioned and assembled into a structure with minimal additional site works (CIDB, 2003).

IBS is a new trend introduced to promote systematic construction process and to reduce the dependency on foreign workers. IBS in Malaysia has begun in early 1960's and the government had started the first project on IBS at Jalan Pekeliling, Kuala Lumpur. This idea is out into view when Ministry of Housing and Local Government of Malaysia visited several European countries and evaluate their housing development program (Thanoon *et al.,* 2003). This project comprising 7 blocks of 17 storey flat with 3000 units of low-cost flat and 40 shop lots. Besides, the earliest housing development project using IBS was Taman Tun Sardon, Penang (Din, 1984). However, recently the use of IBS as method construction in Malaysia is evolving parallel with the establishment of more and more local manufacturers in the industry.

From the research that have been carried out by CREAM's team there are barriers of IBS implementation in Malaysia which categorized as follows; standardization and quality issues, consumer acceptance, professional perception, process and supply chain, technology, training and education, finance and costing, incentive and communication related issues. Despite these barriers, IBS is still predicted to lead Malaysian construction industry towards modernization and globalization (Mohamad Kamar *et al.*, 2009).

The endorsement of IBS Roadmap 2003-2010 in Malaysia expressed the seriousness of the government and the urgency of IBS implementation. The content of this roadmap is focused towards achieving the industrialization of the construction sector and the longer term objective of Open Building Systems (OBS) concept (CIDB, 2007).

One of the important things that stated in the roadmap is the introduction of Modular Coordination (MC) concept. MC has been universally adopted as an internationally agreed system of dimensioning in the building process, especially in connection with industrialized systems. MC can be explained as all dimensions of building components and dimension of space are expressed in terms of a basic unit which is called a module. This modular dimension can be easily fitted into the dimension of plan layouts and building and economy designed can be achieved. In addition, with this system conflict of communication also can be avoided and a large variety of elements will permit flexibility in architectural planning (Trikha and Ali, 2004).

This system will ensure the components can be fitted together without cutting or extending, even though the components may come from different manufacturers. Besides, with the structural coordination, the standardized units can be used in multiple units and can be produced in large numbers in the factory. Thus, the advantages of standardized mass may reduce production time, the use of materials with less wastage and better quality control. The guide and list of modular coordination were explained in Malaysian Standard, MS1064:2001 which is published by Department of Standards Malaysia (Trikha and Ali, 2004).

2.2.1 The Advantages of Precast System

There are some advantages of the precast concrete system:

- The economy in the use and cost of auxiliary materials (formwork, scaffolding) – considerable economy in cost can be achieved by the repetitive use of moulds to produce the precast components.

- Reduced building time – the total building time can be shortened by starting the building activity on several parts simultaneously.

- Adaptability to social circumstances – the number of unskilled workers from foreign countries can be reduced and thus reducing the social and political problems.

2.3 Joints and Connections

The design and construction of joints and connections are a vital part in precast concrete structures due to their role to transmit forces between structural elements to provide stability and robustness (Elliott, 2016). There is distinguishing meaning between a joint and a connection that should be understood. A joint is an action force such as tension, shear or compression that takes place at the interface between two or more structural elements which is contributed by an intermediate medium like rubber, steel, mortar, epoxy, etc. The capacity of joint is greatly influenced by how much these materials differ from concrete (Elliott, 2016). The right philosophy for a connection is the total construction including the ends of the precast concrete components that meet at it (Elliott and Jolly, 2013). The relation between joint and connection is some of the joints are used to form the major connection like beam-to-column connection and can be more understood through Figure 2.1.
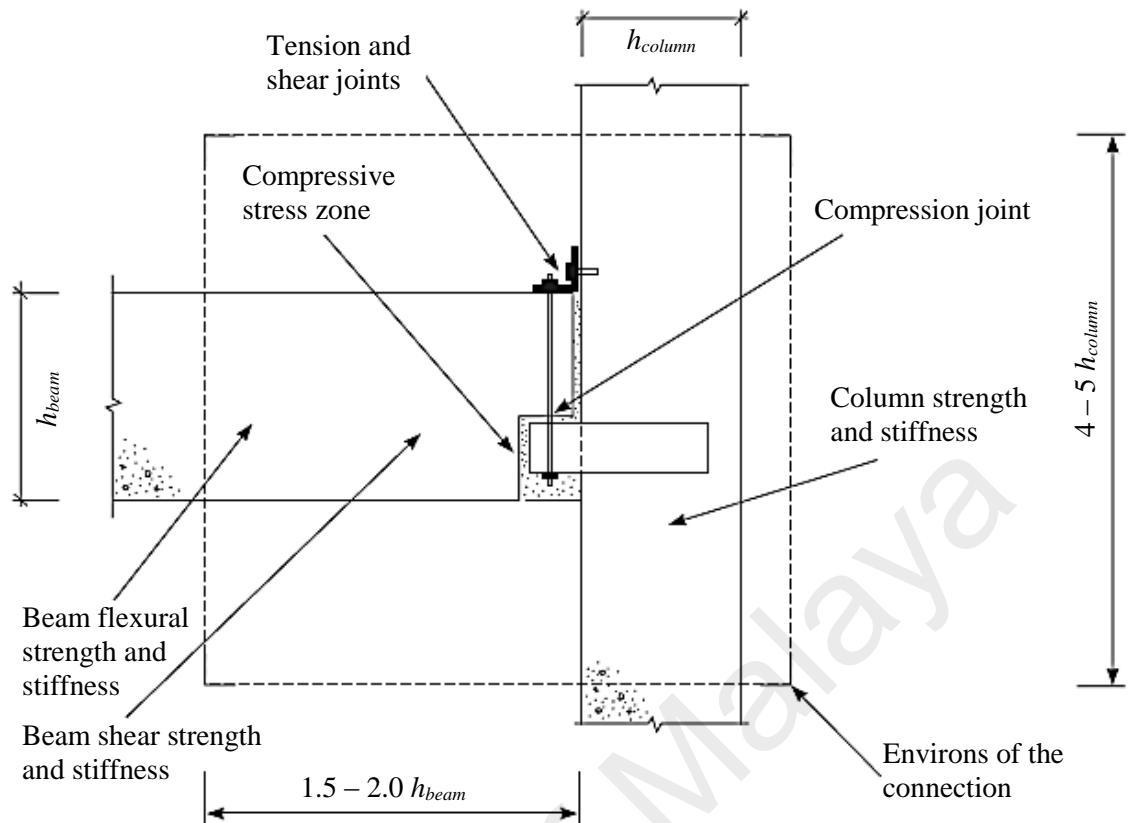
Figure 2.1: Definition of joint and connection (Elliott, 2016)

According to Trikha and Ali (2004), the important structural requirements to be complied with are stated as below:

a) The connection should resist the ultimate design forces in a ductile manner.

b) The overall integrity of the structure and its robustness must be ensured.

c) The connections must be durable and fire resistant.

d) The visual appearance of the connection must be acceptable and aesthetic.

e) The connection also must be simple to ensure it is more economical.

2.4 Moment Resisting Connection

Generally, the moment resisting connection (MRC) has the capability in transferring bending moment to some degree. However, the drawbacks of MRC are that it is difficult to construct and costly. One of the methods used to achieve the capacity in MRC is

grouted or infill concrete joints. According to Elliott (2016) the basic principle of MRC is shown in Figure 2.2. There are some purposes to use these connections:

1) Stabilize and to increase the stiffness of portal and skeletal frames

2) Reduce the depth of flexural frame members

3) Distribute second order moments into beams and slabs, and hence reduce column moments

4) Improve resistance to progressive collapse
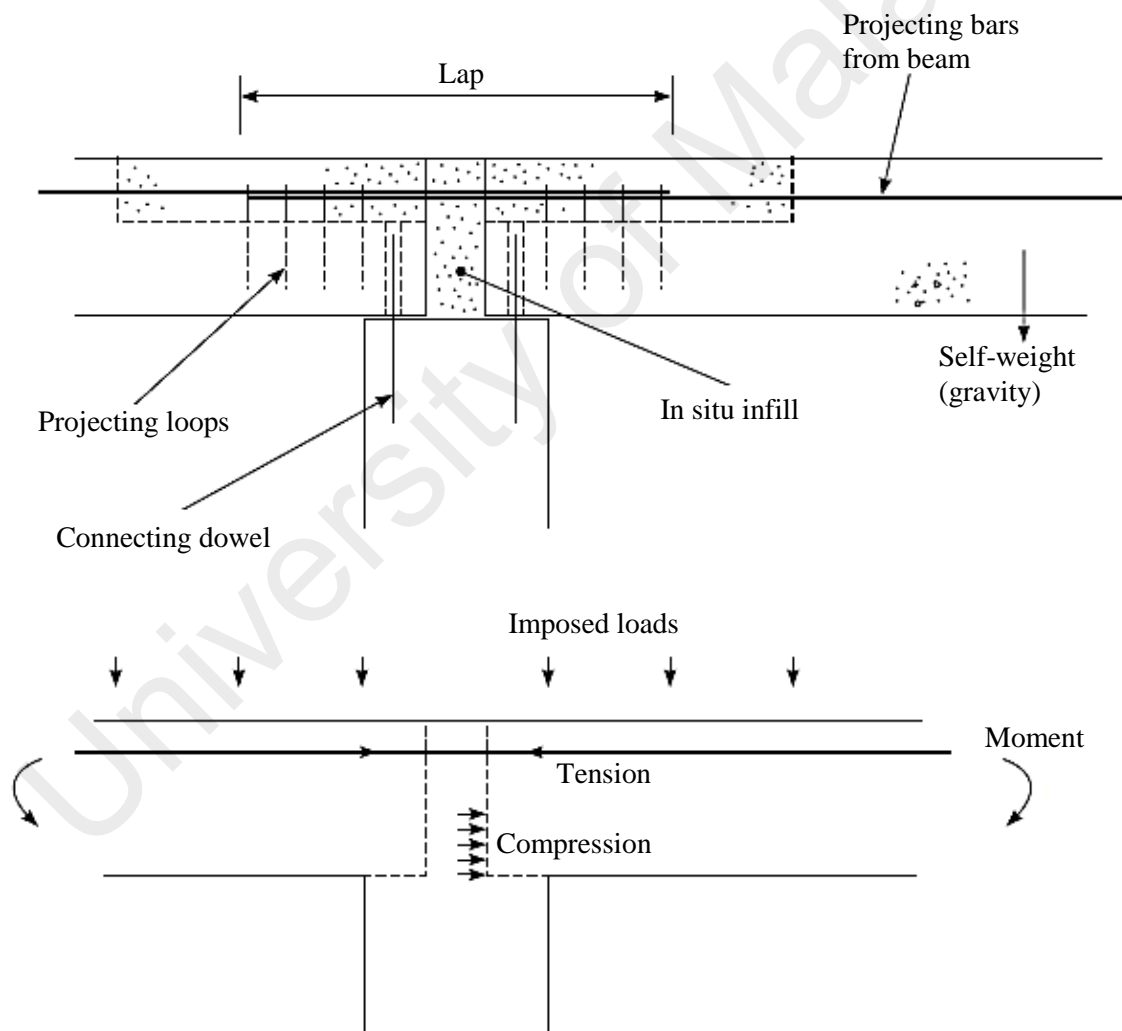
Figure 2.2: Principles of moment resisting connections (Elliott, 2016)

2.5   Review of Previous Investigations

2.5.1   Corbel

There are a few types of beam-to-column connections that applied in the industry currently. The most popular connection in this industry is beam supported on corbel. However, this type of connection is not preferred by architects due to the appearance of corbel is not aesthetical. Therefore, the way to improve it is to hide part of that corbel as shown in Figure 2.3 below.
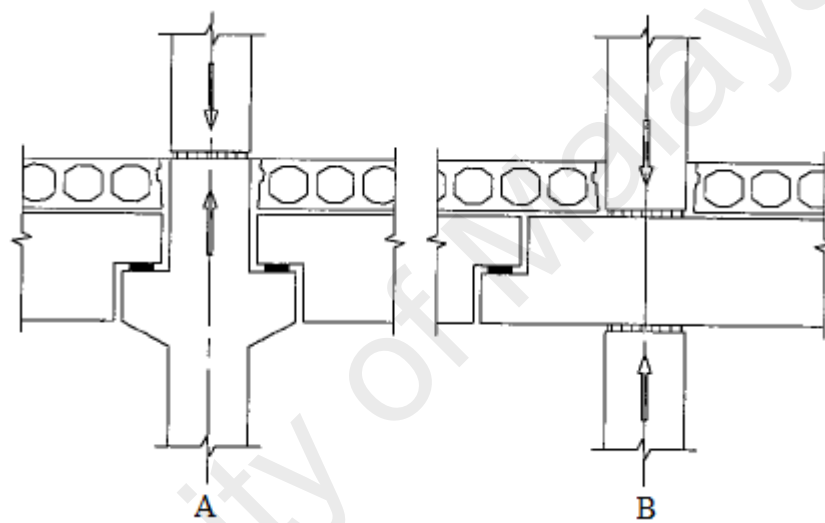
Figure 2.3: Two alternative solutions for beam-column connection. Solution A will perform better than B (Elliott, 2008)

A corbel is a short cantilever projection from the face of column (or wall) which supports a load bearing component. Normally in construction corbel is a pinned connection which is it can transmit shear and axial loads. However, this type of connection can be a ductile connection with some modification such as makes it a hybrid connection or combine with the wet joint. Usually, the beam is a single vertical dowel which is either a waiting bar cast into the corbel or site fixed into a hole. The diameter of the dowel is between 16 and 25mm typically, with the dowel hole is 35 to 50mm. Then the hole will be filled with non-shrink grout from the top (Elliott and Jolly, 2013). Figure

2.4 below shows beam supported on corbel. Meanwhile, Figure 2.5 below shows a graphic illustration of the load transfer through a corbel given by Elliott (2008).

Corbels are simple and direct connections, appropriate to industrial structures
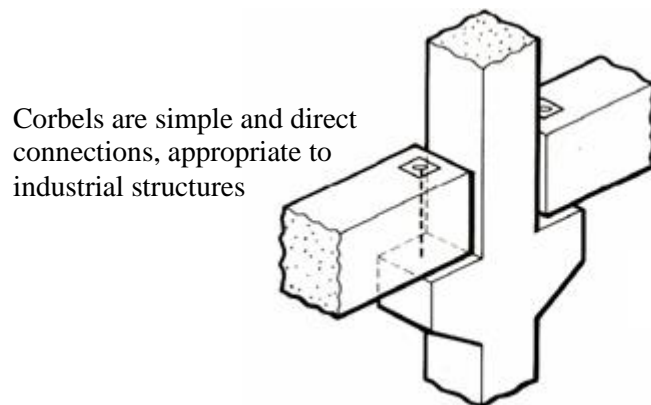
Figure 2.4: Beam supported on corbel with dowel bar (Richardson, 1991)

Δ = total manufacturing and construction

P

θ

$\theta \approx 45^0$

ϕ

α

$\alpha \approx 60^0$
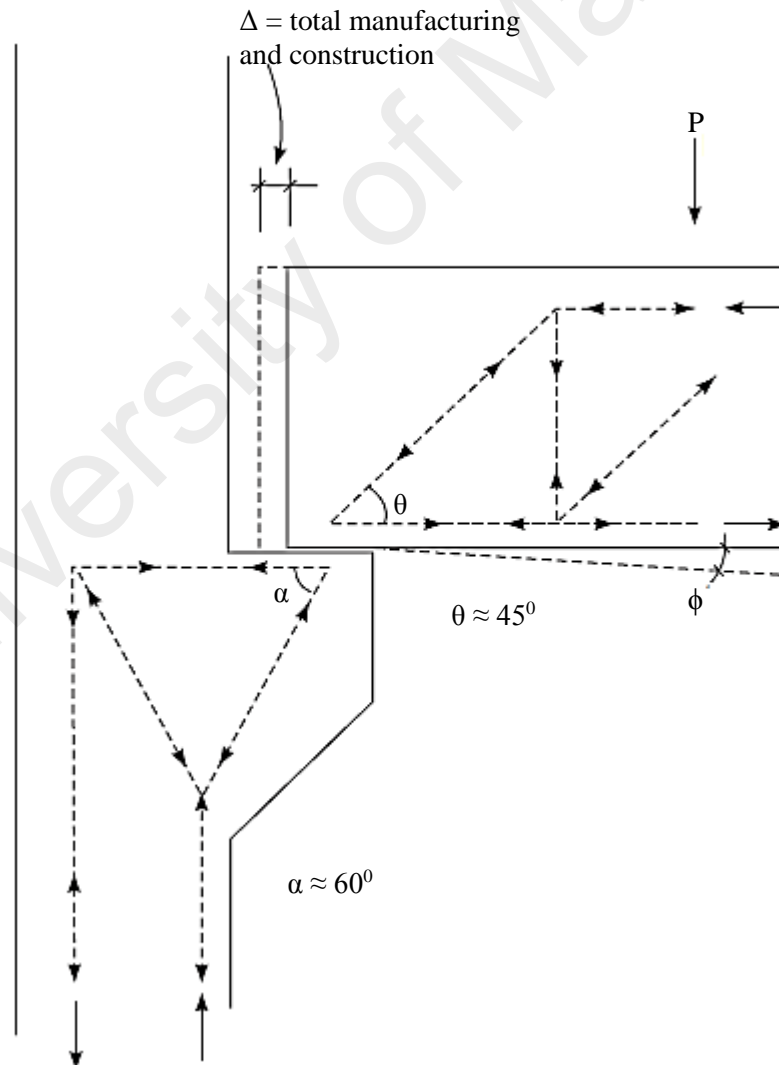
Figure 2.5: Shear force transfer between beam and column through beam and corbel (Elliott, 2016)

2.5.1.1 Failure modes of corbel

The failure modes of corbel are listed as follows based on extensive test program and shown in Figure 2.6 (Park and Paulay, 1975):
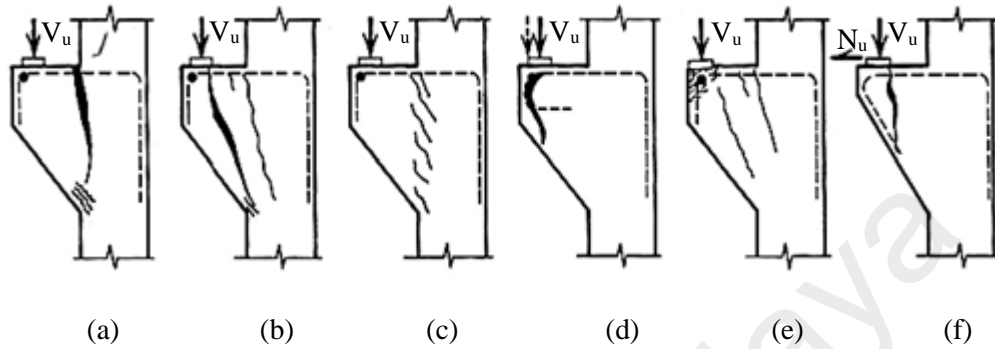
Figure 2.6: Failure modes in corbels (Park and Paulay, 1975)

a) Flexure tension – excessive yielding of flexural reinforcement due to crushing of concrete at the sloping end of the corbel.

b) Diagonal splitting – it occurs after the formation of flexural cracks along the diagonal compression strut and the cause is due to shear compression.

c) Sliding shear – a series of short and steep diagonal cracks and may cause the sliding shear failure when these cracks interconnect.

d) Anchorage splitting – when the applied load is too near to the free end of a short cantilever and area with poor anchored flexural reinforcement. The unintended eccentricity is also may be the cause and due to rotating end of a freely supported beam.

e) Crushing due to bearing – the bearing plates used are too small or very flexible, or when the corbel is too narrow causing crushing of concrete underneath.

f) Horizontal tension – it arises when a horizontal force N_u is present other than the gravity load V_u. The dynamic effects on crane girders, by shrinkage, creep

or temperature shortening of restrained precast concrete beams that attached to the corbel may also be the factor.

2.5.2   Hybrid Connection

Recently, most of the researchers used a combination between steel and concrete as a connection known as a hybrid connection. The term of hybrid connection can be described as mixed construction which is used to combine with other building media such as cast in-situ concrete, steelwork, masonry and timber (Elliott and Jolly, 2013). Figure 2.7 below shows the detailing of the beam to column connection using corbel and angle cleat to increase its performance.
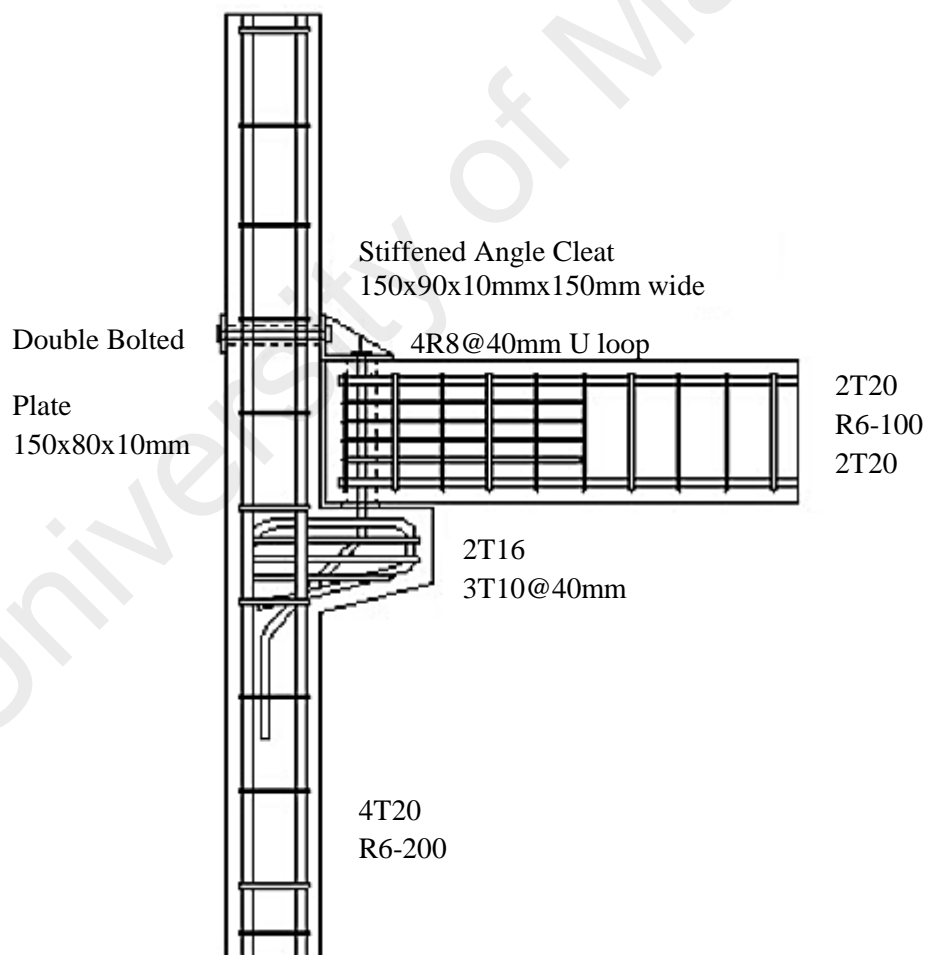
Figure 2.7: Hybrid connection (Abd. Rahman *et al.,* 2006)

Abd. Rahman *et al.* (2006) proposed a hybrid steel concrete connection as shown in Figure 2.7, the use of angle cleat in precast concrete simple beam-to-column connection had increased the load resistance of the entire connection in the test. According to Abd. Rahman *et al.* (2006), as load resistances of specimens had been improved, moment resistances could also be enhanced. Besides, when more bolts are used in the connector, the moment resistance also can be improved. Therefore, the use of stiffened type connector could still attain higher moment resistance. However, it is found by adding steel angles in the specimen did not significantly reduce the ductility of connection. Hence, adding steel angle cleat in the tested simple connection had improved the performance of the entire connection.

2.5.3 Other Types of Connections

Since 1990, 24 full-scale tests were conducted on a wide range of beam-to-column connections as shown in Figure 2.8 and 2.9 respectively. These types of connections have proven satisfactory for semi-rigid designs. The welded connector is a modified Cazaly hanger where the cantilever beam is replaced by a deep narrow plate and the steel. The no hooked-end reinforcing bars welded to the either side of the plate. While, the billet connector is based on the conventional steel haunch but without reinforcing bars welded to the sides of the box section (Elliott *et al*., 1998). Parastesh *et al.* (2014) developed new interior and exterior ductile moment-resisting precast connection which showed suitable for reinforced concrete frames located in high seismic zones.

The most important conclusion is the double-sided connections achieved full capacity while the single-sided connection is limited by the strength of the connector itself, as tie steel is not fully effective and would normally be classified as pin-jointed. In addition, both of Figures 2.10 and 2.11 are cleat and sliding plate types of the beam to column connector respectively that also normally have been used in precast construction

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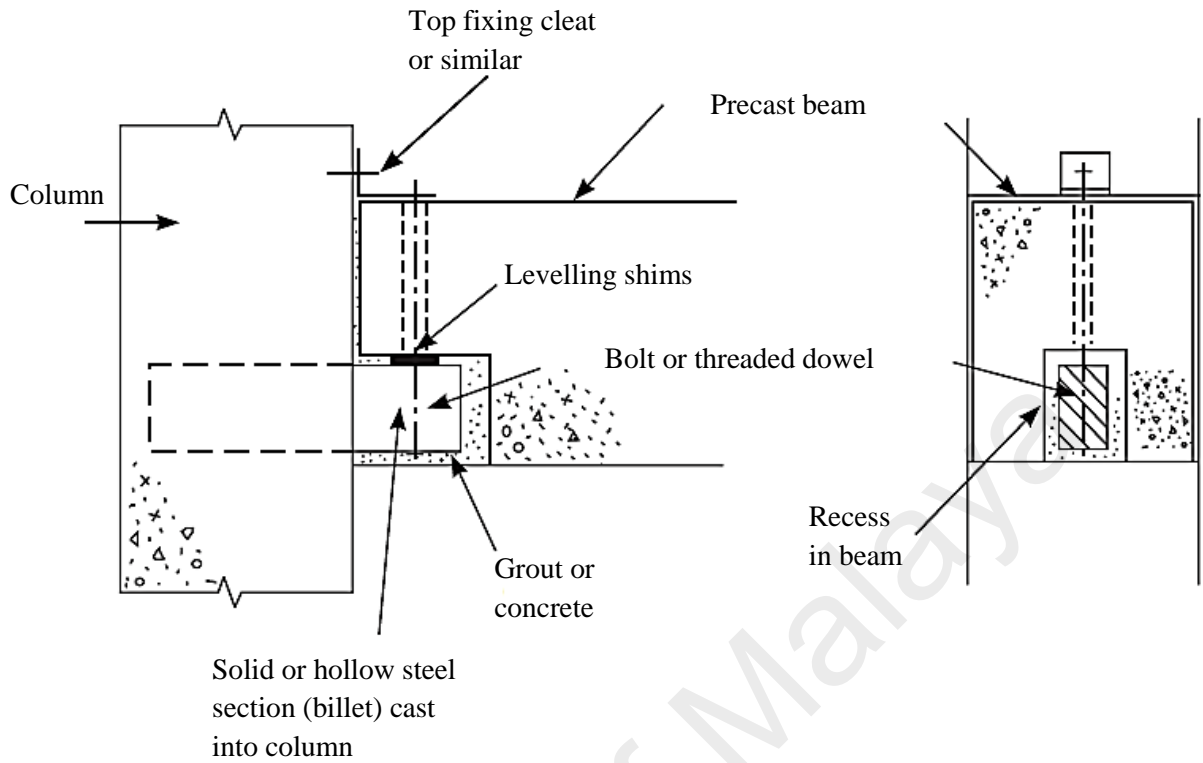
Figure 2.8: Billet beam-to-column hidden connection (Elliott *et al.,* 1998)
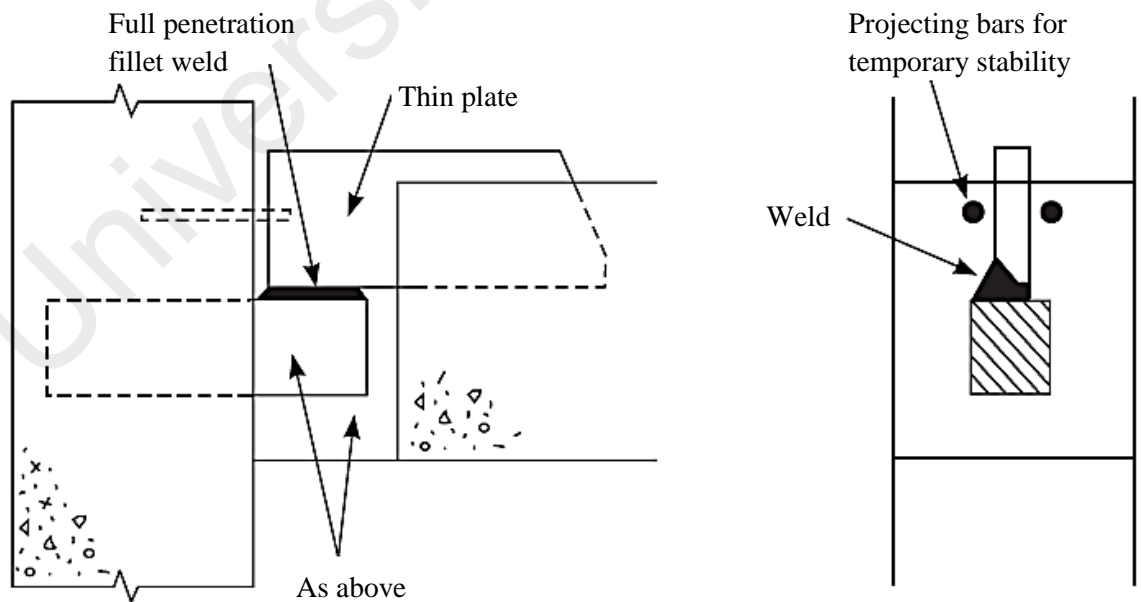
Figure 2.9: Welded plate beam-to-column hidden connection (Elliott *et al.,* 1998)
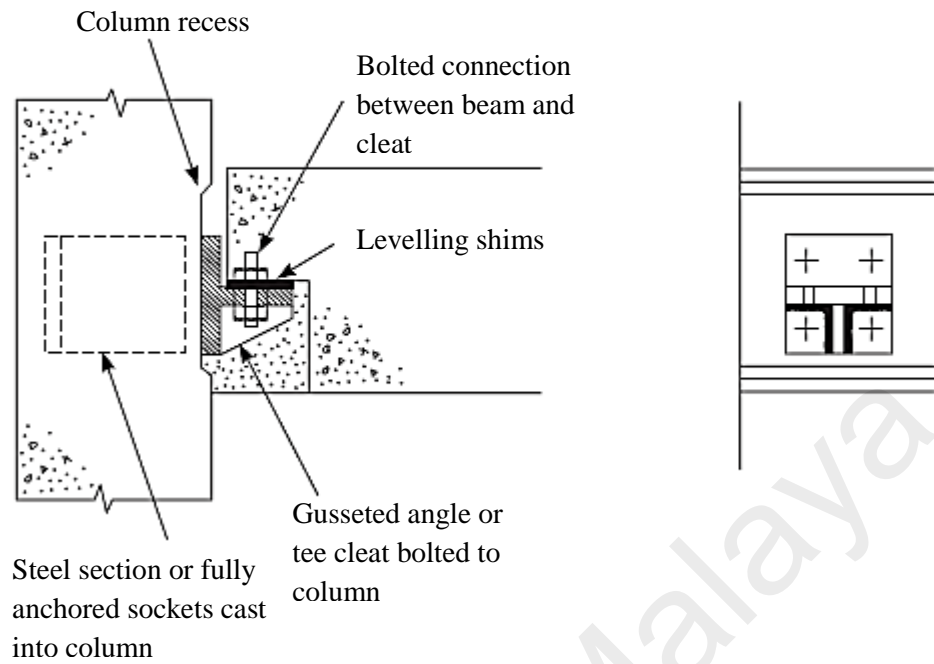
Figure 2.10: Cleat beam-to-column hidden connection (Elliott, 2016)
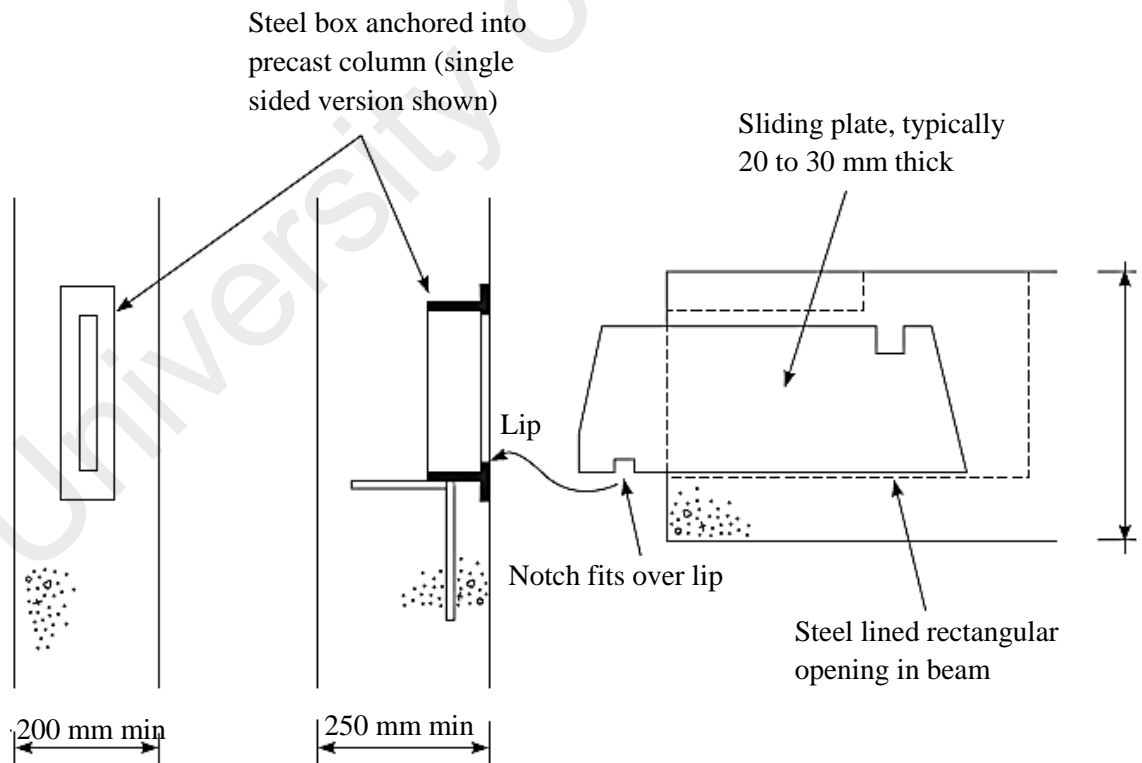
Figure 2.11: Sliding plate beam-to-column hidden connection (Elliott, 2016)

2.6 Precast Concrete Connection Elements

In regard to the structural behaviour, the most essential property is the ability of the connection in transferring the forces. This ability should fulfill the needs at ultimate state as well as in serviceability limit state. Each connection must have sufficient deformation capacity and ductility and also secure the intended structural interaction (Elliott, 2008). For the unbraced frame, the horizontal force resistance is provided by moment resisting frame action as shown in Figure 2.12.

Rigid joints   Column effective lengths, $l_e = 1.6\, l_o$
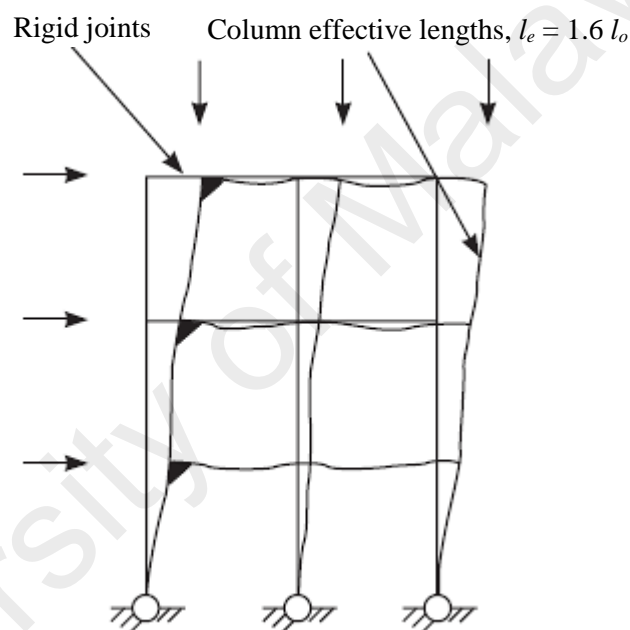
Figure 2.12: Frame action (Elliott, 2016)

2.6.1 Full and Partial Continuity

Normally, the connections in precast concrete structures will give a certain restraint although they are designed as simply supported. The connections can be classified as pinned, semi-rigid, and rigid depending on the connection stiffness. They have finite stiffness and moment capacity but are usually weaker than the connected elements (Elliott *et al.,* 2004; Gorgun, 1997). A connection that having small stiffness can be classified as pinned otherwise it is classified as fixed. Usually, beam-to-column connections are

designed as pinned or fully rigid, but in precast concrete, they are always semi-rigid with partial strength and contain some amount of rotational stiffness (Fatema and Islam, 2006).
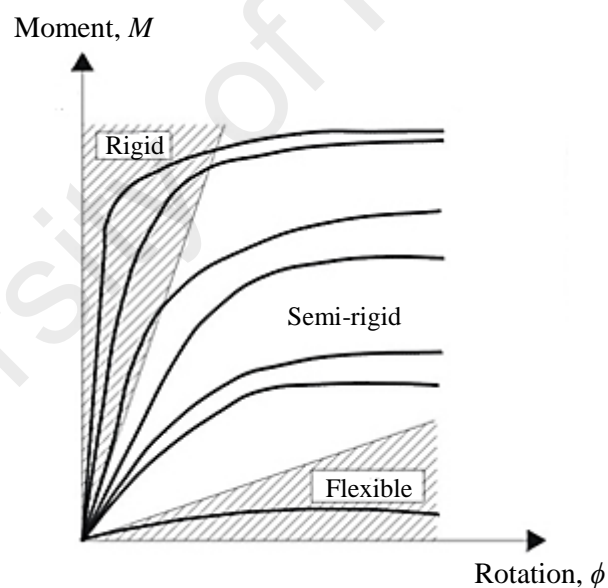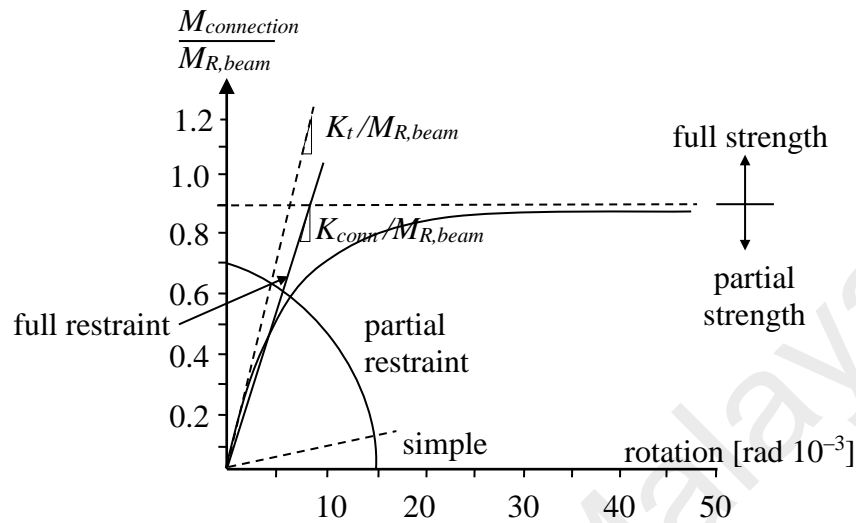
Figure 2.13: Definition of moment versus rotation *M-φ* parameters for connections (Elliott, 2008)

Pinned connections are able to transfer shear only, so this type of connection is applicable for non-sway frames where the horizontal or notional load are resisted by another alternative like bracing or shear walls. While rigid connections can transfer significant moments to the column and it is more applicable for sway frames in terms of

stability and resisting lateral loads. However, this type of connections is more complicated as it is difficult to gain full continuity across joints especially in the soffit and exterior sides.

Semi-rigid connections can fall between simple and rigid connections and often arise in precast construction. For example, bolted connection is often considered as pinned connection, but it could be treated as semi-rigid depending on the joints. Figure 2.13 shows how connections can be classified as fully rigid, semi-rigid, or pinned, depending on its moment-rotation characteristics relate to the strength and stiffness of the structural elements.

2.6.2   Strut and Tie Method

Connection zones in precast concrete elements are subjected to high concentrated forces. All these forces are spread across the sections into wider stress distributions. Then, cracks will appear in these zones if the concrete tensile strength is reached. Therefore, the detailing must be proper to avoid from damages. It is quite normal if cracks happen when the structure is loaded. However, these cracks can be controlled by providing sufficient amount and arrangement of reinforcing steel.

One of the appropriate tools to design the connection zones and check the equilibrium is strut and tie methods. The concept of this method is it consist compression members (struts) and tension members (ties) as shown in Figure 2.14 below. This method also will help designers to understand the behaviour of connection by understanding the flow of forces through the structural connection. Basically, strut and tie is based on the theory of plasticity and gives theoretically a lower bound solution (Elliott, 2008).
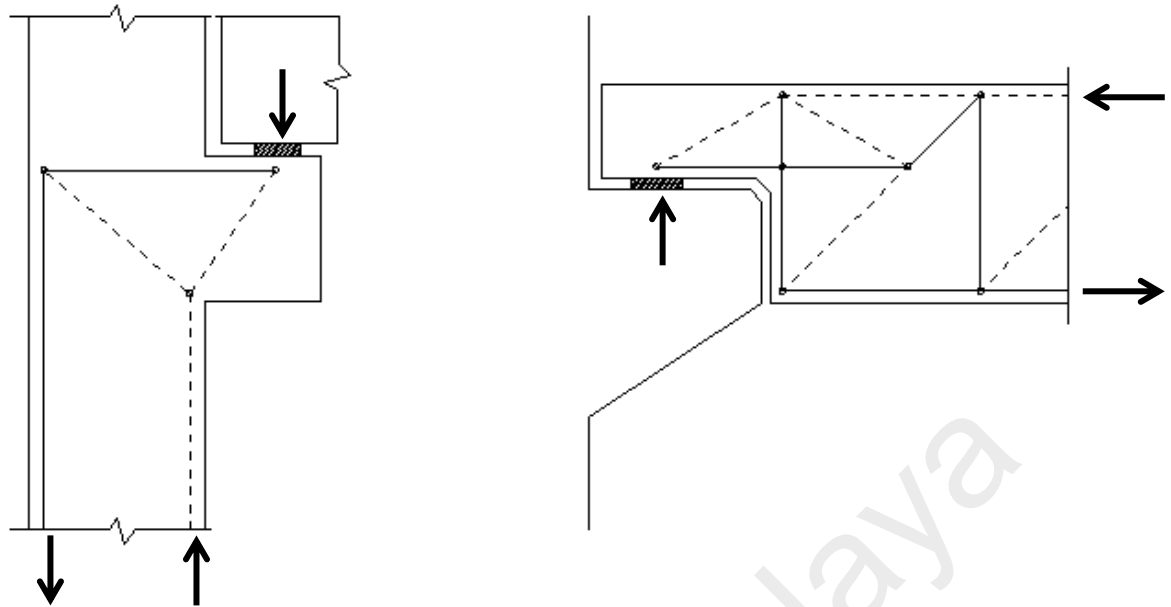
Figure 2.14: The strut and tie force systems (Elliott, 2008)

The member in strut and tie systems are checked with regards to their strengths. As long as the chosen stress field is in the equilibrium with the applied load and there are no critical regions to overstress their strength, then the stress field is theoretically applicable (Elliott, 2008). This diagram is useful to determine the placement of strain gauges in crack monitoring.

2.6.3   Tensile Force

The connections must be presumed that the section is cracked if they are designed to be tensile resistant. This tensile force must be resisted by certain arrangements, e.g. tension bars are fully anchored at both sides of the joints. The anchorage can be designed according to standard methods and it should be linked to the main resisting system of the components to ensure that a continuous force is achieved.

Basically, the connections with tensile capacity are part of tying systems for the structure and should provide structural integrity and thus can avoid progressive collapse. This type of connection must be designed to have ductile behaviour, so premature brittle

failures must be avoided. Therefore, enhanced requirements of anchorage may be justified at the ultimate limit state (Elliott, 2008).

2.6.3.1 Anchor bar

Since the tensile force is transferred successively along the anchorage length so this is the favourable way to anchor connections details. However, no anchorage is perfectly rigid, but the bond is transferred between the anchor bar and the surrounding concrete. The bond stresses that distributed along the steel and concrete interface are not uniform and the slip varies along the anchorage length (Elliott, 2008).
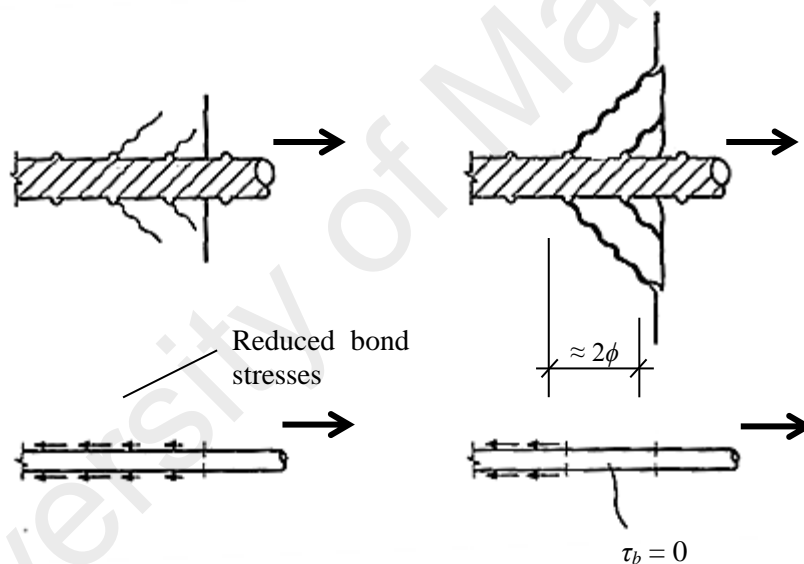
Figure 2.15: Local bond failure near the free edge because of inclined cracks (Elliott, 2008)

In the design, it is generally assumed that the bond stress along the anchorage length reaches the bond strength. However, this assumption is a simplification of the real behaviour. Variety of failure modes could happen depending on the actual detailing and material properties. According to Elliott (2008) the anchorage failure may be due to the splitting of the concrete or shear failure that develops along the interface as shown in

Figure 2.15. In case of short anchorage, a pull-out failure could occur before yielding of the steel is reached.

2.6.4 Bearing Type and Material

Bearing is one of the most important aspects in precast concrete elements. The main purpose in designing bearing material is for vertical and horizontal loads and rotation and lateral movements. The size of bearing area is generally determined by the size of concrete elements, erection tolerances and architectural considerations. There are a few types of bearing that normally used in precast concrete constructions as shown in Figure 2.16. To choose the type of bearing material is depending on the design requirements which mean how the function of that building e.g. loading and type of connection. The lists of the bearing are as follow (Elliott, 2008):

- Dry bearing – precast to precast or precast to in-situ concrete.
- Dry packed bearing – elements are located on thin shim (3 to 10mm thick) and the resulting small gap is filled using semi dry sand/cement grout.
- Bedded bearing – elements are positioned onto a prepared semi-wet sand/cement grout.
- Elastomeric or soft bearing – neoprene rubber or similar bearing pads.
- Steel bearing – steel plates or structural steel sections.

The design of ultimate bearing stress is based on the cube crushing strength of the weakest of the two or three component materials. The guide is listed as follows (BS8110, clause 5.2.3.4):

a) Dry bearing on concrete – $0.4f_{cu}$

b) Wet bedded bearing on concrete or mortar – $0.6f_{cu}$

c) Elastomeric bearing (called flexible padding) – between $0.4f_{cu}$ and $0.6f_{cu}$

d) Steel bearing – 0.8f_{cu}. For larger bearing plates, the allowable bearing stress fb is given by equation 1.

$$f_b = \frac{1.5f_{cu}}{\left(1 + \dfrac{2b_p}{b}\right)} \tag{1}$$
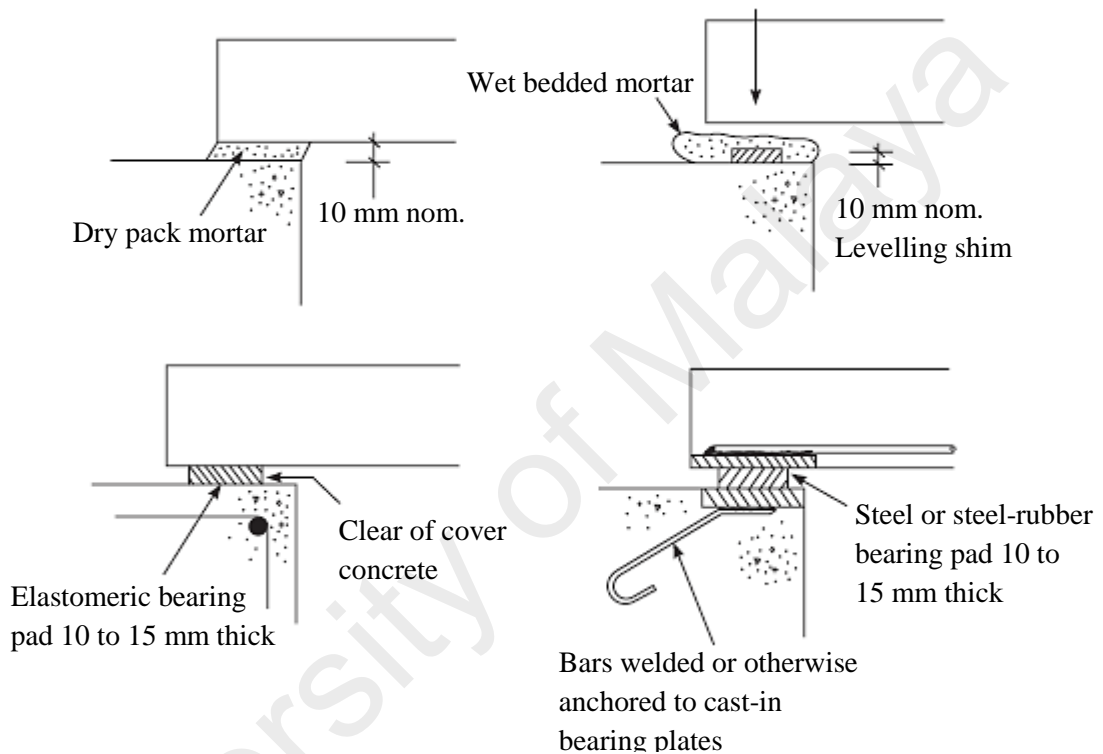
Figure 2.16: Types of bearings (Elliott, 2016)

2.7 Load-Displacement Relationship

The behaviour of connection either ductile or brittle can be determined by the load-displacement characteristics. In some cases, if a structure being excessively loaded, it should be capable of undergoing large deflections at near-maximum load carrying capacity to give warning of failure and thus can avoid from progressive collapse (Paul Leong, 2006).

Ductility is a very important part in design consideration especially if the connection is subjected to seismic loading. The ductility is the ability of the connection to undergo large plastic deformations without a substantial reduction of the force that is resisted (Elliott, 2008). The ductility is often expressed as the ductility factor $\mu = \mu_u$, ultimate deformation / μ_y, deformation when a plastic behaviour is reached (Elliott, 2008). Figure 2.17 below shows the typical load-displacement curve to determine the behaviour of connection.
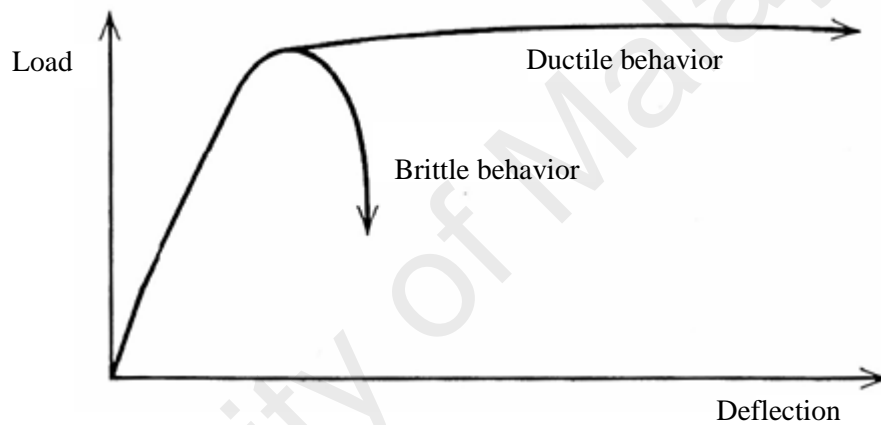
Figure 2.17: Typical load-displacement curve (Park and Paulay, 1975)

2.8 Moment-Rotation Relationship

From the testing of beam-to-column connections, the value of the moment is gained by multiplying the corresponding applied point load with the distance of the point load from the surface of the column. While for rotation, the value is obtained by dividing the corresponding vertical displacement with a distance of LVDTs from the surface of the column (Elliott *et al.,* 2003).
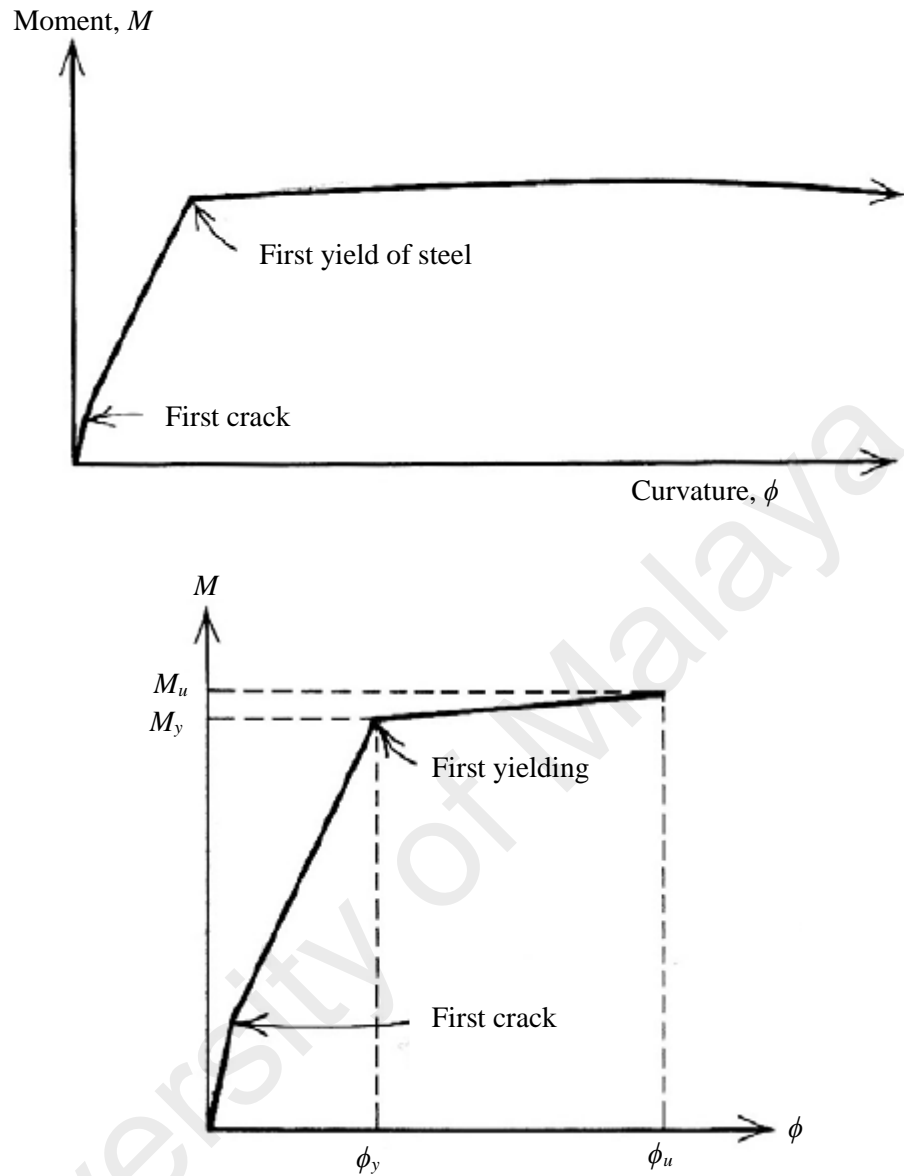
Figure 2.18: Moment-rotation curve (Park and Paulay, 1975)

In addition, to avoid twice counting of rotation, the rotation must not include curvature of beam or column known as influence zone. Usually, the length of zone beyond influence zone is approximately equal to the depth of the adjoining beam or column (Elliott *et al.,* 2003). The typical graph of moment-rotation relationship and the ductility of connection are illustrated in Figure 2.18.

2.9   Summary

Eventually, the behaviour of frames is controlled by the characteristics of the connections. To satisfy the structural requirements, each connection must have the ability to transfer vertical shear, transverse horizontal shear, axial tension and compression, bending moments and also torsion between one precast component with another safely.

Currently, beam-to-column connections such as corbel, billet, and hybrid are applied in the construction industry (Elliott *et al.*, 1998). The most popular connection in this industry is beam supported on corbel. For instance, according to Vidjeapriya and Jaya (2012), the presence of corbels will increase the connection's stiffness. However, this type of connection is not preferred by the architects due to the appearance of corbel is not aesthetically viable. Moreover, the design of corbel connection is normally pinned. Therefore, among the objectives of this study is to develop an MRC that is fast and clean, to construct an aesthetically viable and economical precast concrete connection system for a low-rise. Hence, a grouted connection (hidden corbel) was chosen to be studied.

Hidden corbel is a physical connection relying on bearing, bond, and anchorage. This connection is formed by casting a certain amount of in-situ grout concrete around projecting reinforcement (Elliott, 2016). On the other hand, according to Wahjudi *et al.* (2014) wet workings for precast concrete beam-to-column connections have been demonstrated to have higher ductility and over-strength factors compared to equivalent monolithic specimens. Besides that, causes that provoke nonlinear behaviour in reinforced concrete structures include concrete cracking, the plasticity of steel reinforcement, and relative slippage of flexural reinforcement (Alva and El Debs, 2013).

The proposed connection has many advantages over other jointing connections, as it is simpler and requires no bolting or welding like billet or welded plate. Quality control measures associated with excessive welding may create some inherent advantages to

precast concrete, albeit increasing the project's cost (Ertas *et al.*, 2006). The sizes of beam and column have selected based on the standard size of JKR's building provides the opportunity to investigate and validate the use of Corbel connector as well as validation of theory and technology on the basis of experimental evidence. The experimental data and research findings justify the use of these potential connections and some guidelines for its implementation.

CHAPTER 3: RESEARCH METHODOLOGY

3.1 Introduction

Based on the literature survey, current situations and problems encountered, the suitable method was formed. This chapter explains the methodology used to accomplish the research objectives. Therefore, it is divided into three sections; theoretical approach for semi-rigid connection, the experimental procedure to test the specimens and method of analysis to be used. The first section presents the equations that involved in the theoretical approach to predict semi-rigid connections. Subsequent section explicates the details description of proposed precast beam-to-column connections and test procedure. The third section describes the method of analysis to evaluate moment-rotation characteristics and classification of connections.

3.2 Analytical Study for Semi-Rigid Precast Connection

The analytical model can be an alternative method to predict the semi-rigid behaviour of connections. The theory of analytical modelling was studied by Ferreira (1993) and some of the equations were well developed and are currently used in the construction industry. According to Ferreira, the proposed procedure of moment resisting connections should meet both the strength and stiffness requirements simultaneously. The example of calculations is given in Appendix B.

3.2.1 Theoretical Approach to Predict Semi-Rigid Behaviour (Elliott *et al.,* 2004)

The rotational stiffness, S is defined as:

$$S = \frac{M_{RC}}{\phi_C} \qquad (2)$$

Ultimate flexural strength may be calculated according to the BS 8110 rectangular stress block using the usual notation for rebar and cross section. Providing the continuity bars are fully anchored and wrapped by links, the steel should attain yield strength.

Although the concrete and reinforcement parameters vary widely, it is found that for most cases, $z/d = 0.85$ to 0.95. Nevertheless, there is no such restriction in BS 8110. Thus, the moment of resistance becomes:

$$M_{RC} = 0.87 f_y A_s z \tag{3}$$

Total end relative rotation, ϕ_c arises from two primary deformations, beam-to-column rotation due to the joint opening at the interface and beam end curvature along a plastic hinge length, l_p.

Joint opening at the interface: This is due to elongation of top reinforcing bars. The local rotation, $\phi_c = \delta/d$, where d = effective depth to top bars in the beam. The deformation, δ is equal to the yield strain in the bars times embedment length, $\delta = l_e f_y / E_s$, where l_e is taken as the lesser length over which the stress distribution along the bar is uniform or the length available as defined in Figure 3.1. Then,

$$\phi_c = \frac{f_y l_e}{E_s d} \tag{4a}$$

Beam end rotational deformation: This is caused by the curvature of the beam in the region where the curvature of the beam and also the tensile stress in the top bars are constant as shown in Figure 3.2. There is a concentration of cracks that cause curvature, which is constant, within the plastic hinge length, l_p. Meanwhile, for reinforced concrete (RC) beams, $l_p = h_{beam}$. In precast connections, l_p depends on load path from the centre of rotation, type of connector bearing, (e.g. corbel, billet or cleat), and whether the force is transferred to the beam by a cast-in steel plate or by suspension bars. Then,

$$\phi_c = \left(\frac{1}{r}\right)_{cr} l_p = \frac{M_{RC} l_p}{E_c I_{cr}} \tag{4b}$$

where $(1/r)_{cr}$ is the curvature of the beam (or beam plus slab) based on the flexurally cracked section.
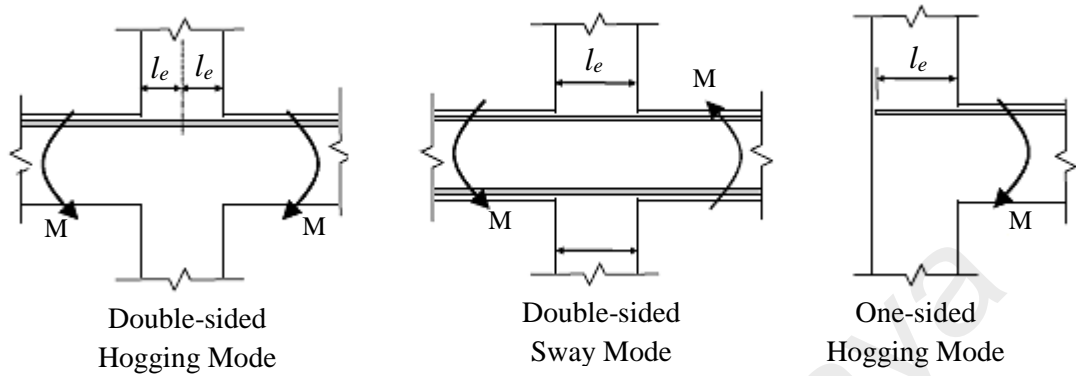
Figure 3.1: Embedment length of reinforcement across columns (Elliott *et al.*, 2004)
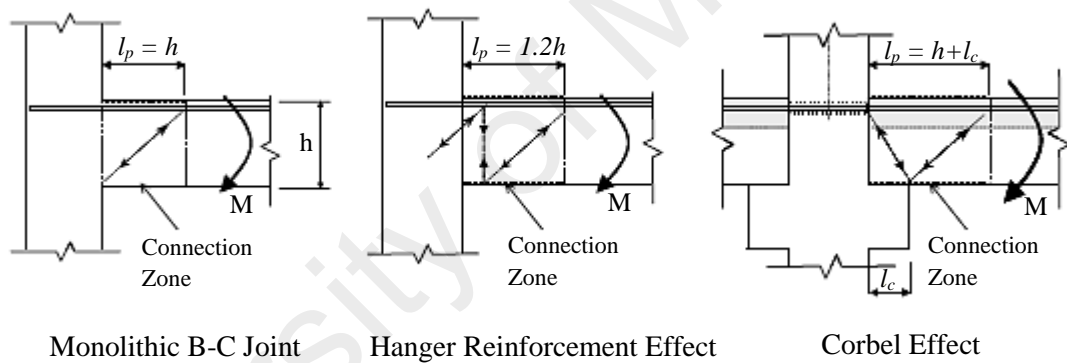
Figure 3.2: Connection zones for types of precast connections (Elliott *et al.*, 2004)

The required moment capacity, M_{ER} and the allowable design moment, M_{ED} for the connector at the ultimate limit state can be obtained from the intersection of $S = M_R/\phi_c$ with the beam-line (from $M_{ER}/M_R = S/S + 2EI/L$) as follows:

$$\frac{M_{ER}}{M_R} = \frac{M_{ED}}{M_D} = \left(1 + \left(\frac{2E_cI_{cr}}{L}\right)\left(\frac{\phi_c}{M_{RC}}\right)\right)^{-1} \tag{5a}$$

Replacing M_{RC} with f_yA_sz and ϕ_c with $(f_yl_e/E_sd) + (M_{RC}l_p/E_cI_{cr})$, equation 5a is rewritten as:

$$\frac{M_{ER}}{M_R} = \frac{M_{ED}}{M_D} = \left(\left(\frac{L+2l_p}{L} \right) + \left(\frac{2E_cI_{cr}}{E_sdA_sz} \right) \left(\frac{l_e}{L} \right) \right)^{-1} \qquad (5b)$$

3.3 Experimental Programme of Full-Scale Connection Tests

3.3.1 Introduction

This section will focus on the methodology of how the objectives for this research that have been listed before can be achieved. The main aim of the full scale single sided (exterior connection) test is to determine the moment-rotation, $M\text{-}\phi$ characteristics and adopt an established method to discover the classification and fixity factor of the connection. Based on these $M\text{-}\phi$ characteristics, it is possible to ascertain hogging moment capacity, rotational stiffness, and ductility of the connection. Besides, the load-displacement relationship, stress distribution and shape deformation can be obtained. Three specimens were tested according to schedule program and subjected to vertically apply bending loads at the free ends of the precast beams. Three specimens were developed for full-scale experimental testing and carried out at the CREAM laboratory to obtain $M\text{-}\phi$ data. The schedule of the testing program is according to Table 3.1.

Table 3.1: Schedule testing program

Connection	Loading Type	Connection reinforcement type	Test reference
Beam half joint with corbel	Reverse load	T1	BHC1
		T2	BHC2
		T2	BHC3

3.3.2 Description of the Proposed Precast Beam-to-Column Connections

The general shape of proposed connection is shown in Figure 3.3. Beam half joint and the corbel were designed based on BS 8110, Part 1: Clause 5.2.7. The design for reinforced concrete corbel was based on strut and tie model which leads to a more accurate representation than by considering the corbel as a short cantilever in bending and

shear. The size of the beam is 300 mm by 450 mm and 1500 mm in length (i.e. 3½ times depth to avoid local boundary effects). While for the column is 300 mm by 300 mm with 3000 mm in total height and a cross section containing 4 no. T25 mm rebars.

However, it is important to check a shear failure will not occur by proportioning the depth of corbel. This precast beam is designed depending on the fabrication, jointing details, delivery and lifting. Designing a precast half beam involves two stages; (i) installation and (ii) service. At installation stage, the rebars area is calculated for self-weight of the beam, floor elements, and wet topping concrete. Meanwhile, at service stage, rebars area for positive and negative bending at the bottom and top of the beam and in-situ infill are calculated for an assumed a fully rigid connection. Details of beam half joint reinforcement are picturised in Figures 3.4 (a) - (b) and 3.5.

Tension reinforcement in the *in-situ* topping is mainly used to resist hogging moment by providing fixity to the column. These tension reinforcements should be fully anchored and lapped (BS 8110: Clause 3.12.8.13, Table 3.27) to ensure full tensile force can be developed at this connection without slippage or failure. Flexural reinforcements were anchored using a 180° hook and 90° bend inside the columns. Nonetheless, no anchorage is perfectly rigid as the bond is transferred between anchor bar and surrounding concrete.

Details of top reinforcement for BHC1, types T1 (180° hook) meanwhile, BHC2 and BHC3, types T2 (90° bend) are presented in Figure 3.6. The connection utilized two 20 mm high yield deformed bars as its top reinforcement.

Hook

Tube

Sleeve

Levelling tolerance (20mm)

Tension bar (2T20)

In-situ concrete

Dowel bar (T16)

Bearing pad (10mm)
(refer Figure 3.8)

Precast half
beam

Shallow corbel
(refer Figure 3.8)

Starter bar

Precast column

100

300

350

300

300

300

4T25

Column cross section
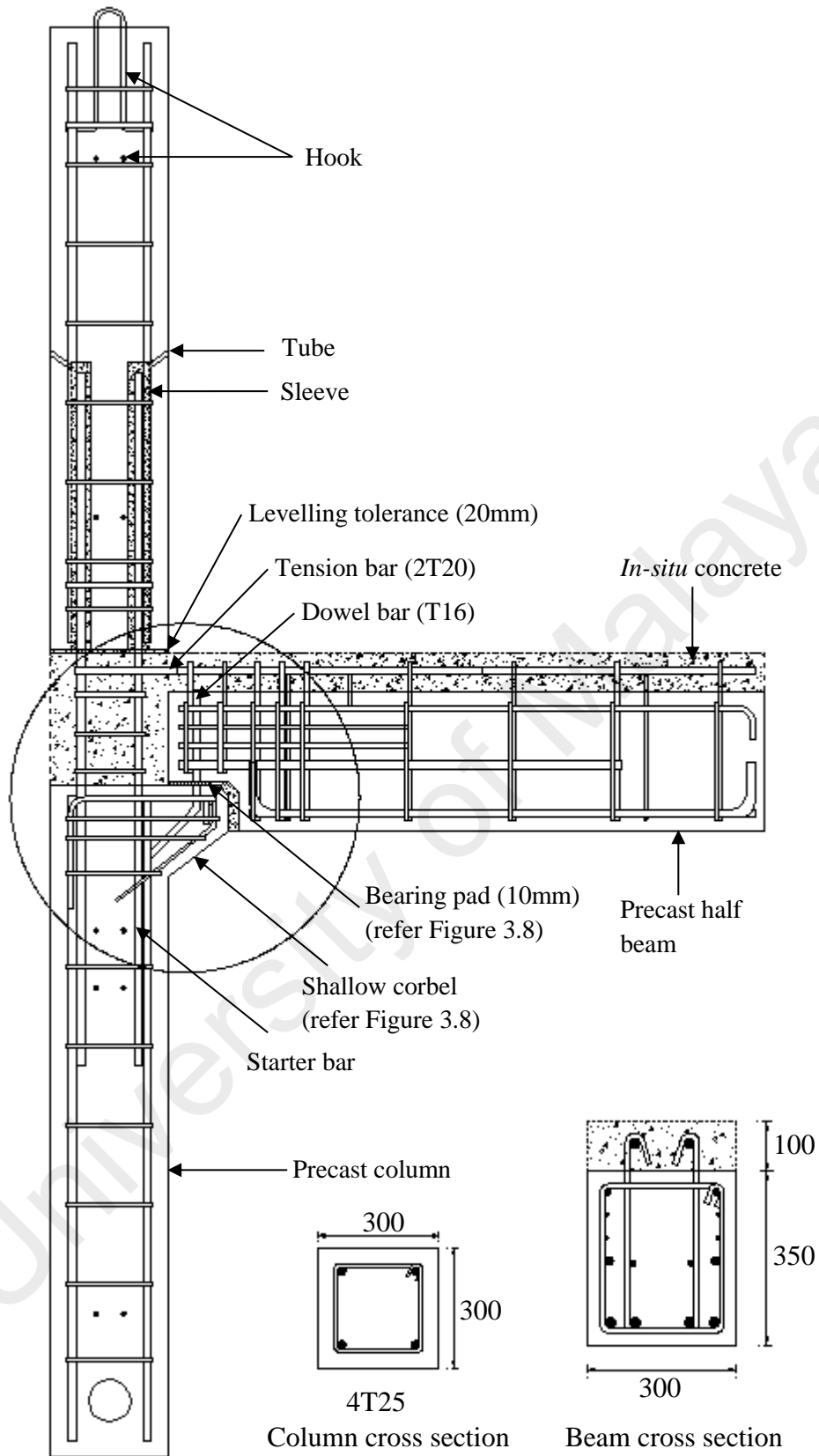
Beam cross section

Figure 3.3: General shape of proposed connection
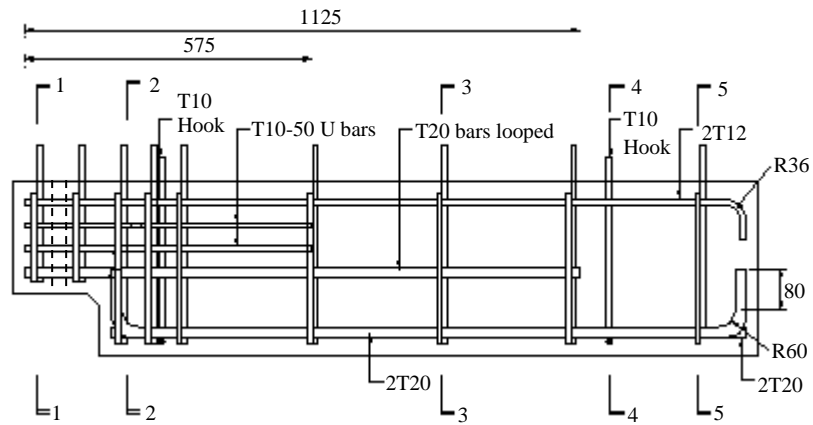
(a) Precast half beam reinforcement
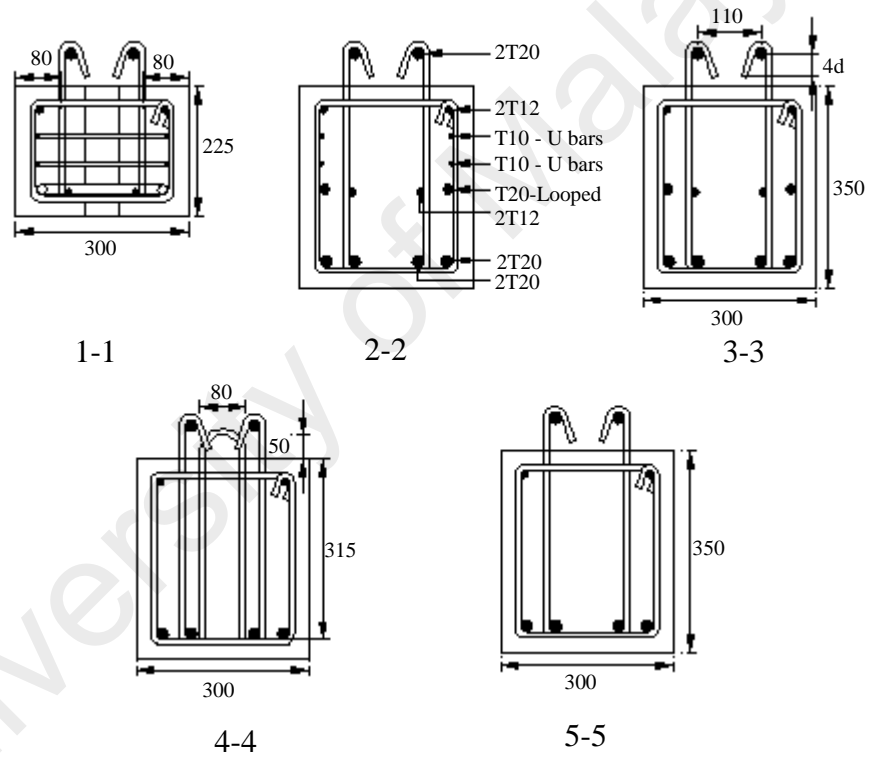
(b) Precast half beam cross section detailing

Figure 3.4: Precast half beam detailing
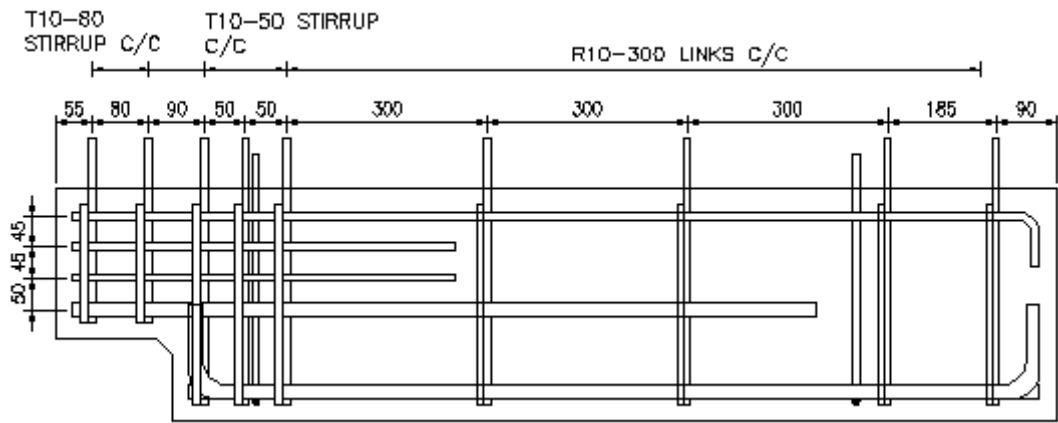
Figure 3.5: Beam half joint shear links detail
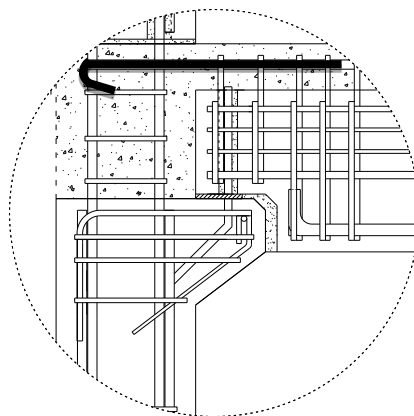
(a) Type T1-180° hook (BHC1).　　(b) Type T2-90° bend (BHC2, BHC3).
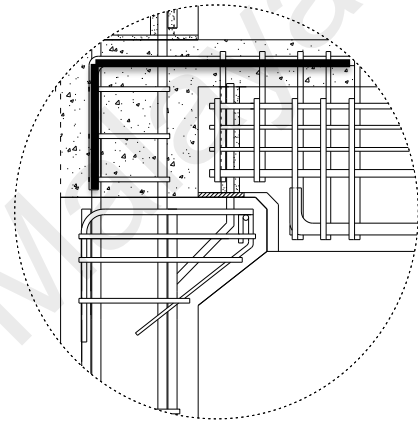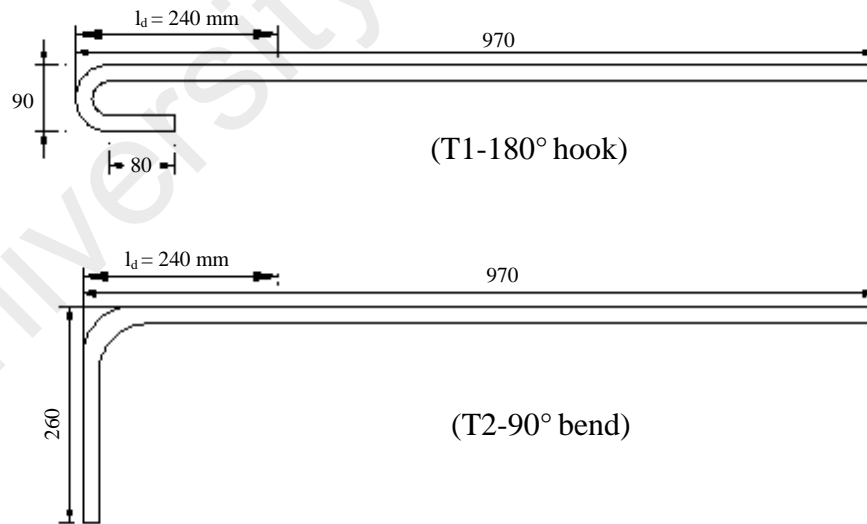
(c) Top reinforcement details.

Figure 3.6: Detailing of tension reinforcement (BHC1, BHC2 and BHC3)

Meanwhile, corbel's function is only as temporary support at installation stage with the size of dowel bar is 16 mm and the system will act as a pinned connection. Nonetheless, after the beam and column were concreted, the whole connection will act as a moment-resisting connection. The shallow corbel utilized two 16 mm high yield deformed bars as its main reinforcement with 10 mm thickness of bearing pad. The details of columns and corbel are shown in Figures (3.7) – (3.8). The designs of components are given in Appendix C - D.
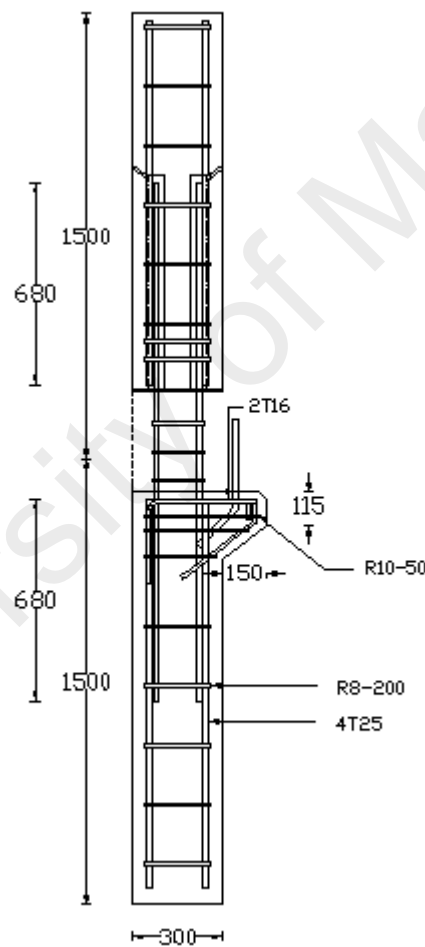
Figure 3.7: Detailing of top and bottom columns
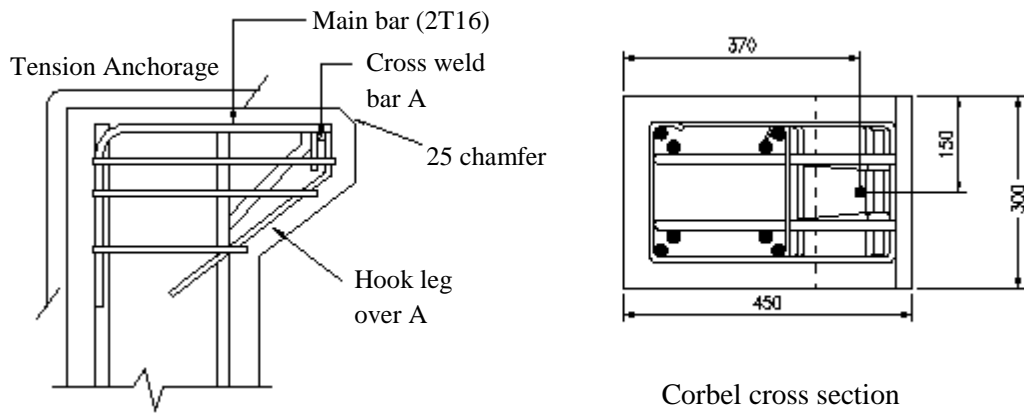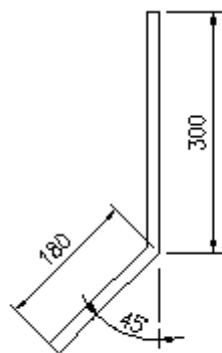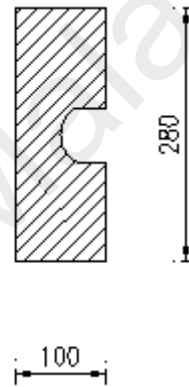
Figure 3.8: Detailing of corbel (refer to Figure 3.6)

3.3.3   Materials

The compressive strength, f_{cu} for the precast components was 40 N/mm² at 28 days. Three precast beam-to-column components were fabricated at Teraju Precast Sdn. Bhd. Factory, Dengkil, Selangor. The infill concrete mix, f_{cui} was designed to have a compressive strength of 40 N/mm² in 7 days in accordance with the requirements of BS 1881: Part 125 (Appendix A). The actual strengths for in situ infill concrete are given in Table 3.2. Slump testing was carried out for every mixing batch to ensure uniformity and workability of the mix. The desired slump measurement is in the range between 60-180

mm. Then, compressive strength tests were performed on two of 150 mm size cubes to give the mean value for analysis purpose.

High yield- steel grade 500 N/mm^2 was used as main reinforcement for all precast columns, corbels, beams and tension reinforcement. The actual strengths for axial tension rebar testing are given in Table 3.3. Whilst, mild steel grade 250 N/mm^2 was used to resist shear forces in all precast columns and beams. Meanwhile, SikaGrout-215 was used to fill the dowel and column sleeves in jointing precast concrete connections. Infill concrete cubes and axial tension rebar testing provided the mean values to determine the strength of in-situ infill concrete, f_{cui}, and rebars, f_y, as given in Table 4.1.

Table 3.2: In situ infill concrete actual strength, f_{cui}

Connection	In situ infill concrete actual strength, f_{cui} (N/mm^2) at testing day	
	Cube 1	Cube 2
BHC1	40	42
BHC2	45	48
BHC3	53	59

Table 3.3: Axial tension rebar actual strength, f_y

Connection	Bars yield actual strength, f_y (N/mm^2)	
	Rebar 1	Rebar 2
BHC1	542	537
BHC2	595	589
BHC3	595	589

3.3.4 Testing Frame Setup

The assemblies were constructed at CREAM Laboratory and according to specifications drawing. The erection of real construction was imitated closely as much as possible in the manner indicated in Figure 3.9.

First, precast bottom column was lifted vertically using a crane and placed into a steel column shoe. The column shoe was restrained to the strong floor using holding down bolts. Next, beam unit was placed on the corbel with a bearing pad (10 mm thick) above it. At this stage, dowel functions to fix beam's position. Then, top bars were tied up to shear links as the jointing. Plywood was used to prepare formworks in a joint concreting or grouting process. The formwork was set up according to the normal practice and techniques used on site as well as concrete manufacturers. Cross woods between two formworks were employed to restrain it from excessive expansion of concrete. After the concrete is ready, the formworks were removed to prepare the top column joint. The precast top column was then raised, passing through top sleeves of the column. Finally, construction was completed by filling column sleeves using grout, in accordance to mix proportions. The beam and column were coated with white emulsion paint to assist the detection of cracks in the concrete under load.
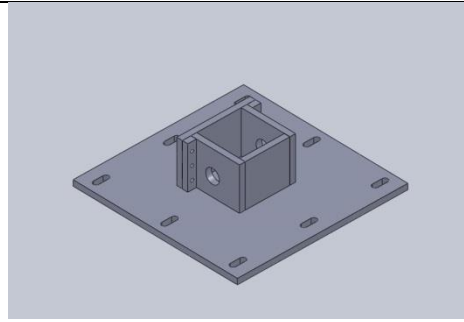
(a) Column shoe to hold bottom column
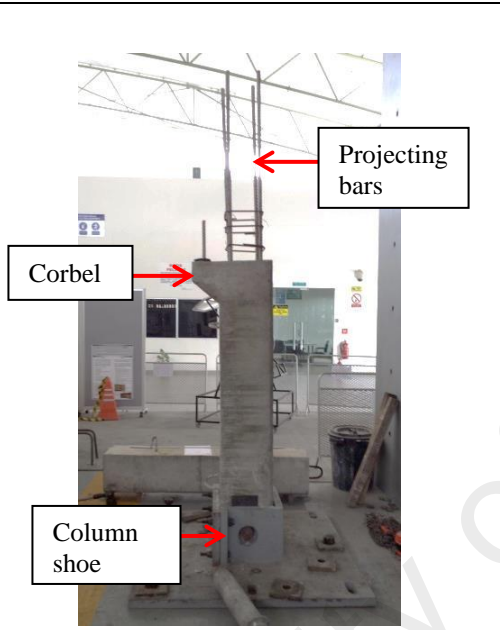
(b) Bottom column set up into column shoe	(c) Half beam placement on the corbel
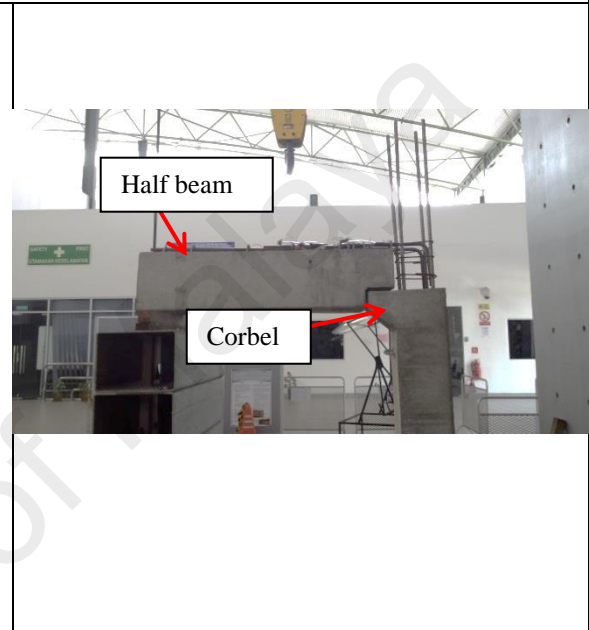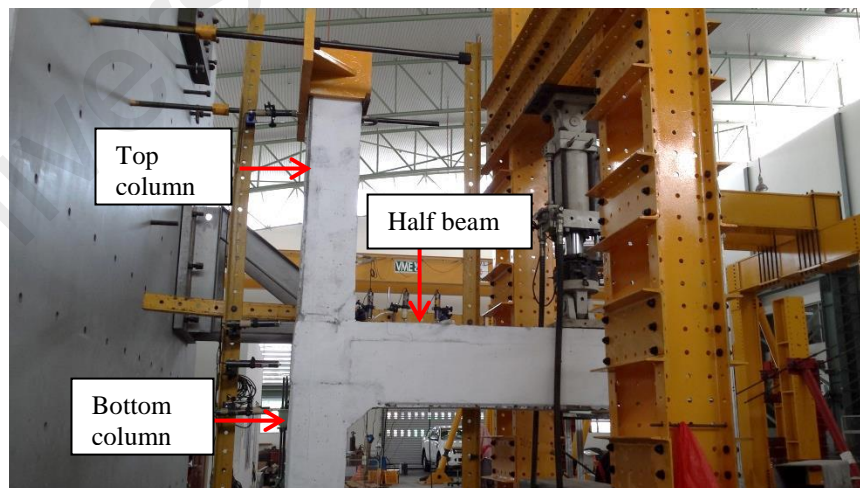
(d) Complete erection of components with top column holder

Figure 3.9: Erection step for specimen

There are two types of test rig which are using tie back steel frame (BHC1, BHC2) and strong wall (BHC3) as shown in Figures (3.10) – (3.11). The strong wall setup is used to minimize the top column rotation. The precast column was first restrained in position with no rotation at top and bottom supports. The top column is tied back to the test rig frame by steel rods on both sides of the column (in plane) for the first test rig (Figure 3.10). Besides that, the frame was anchored to the strong floor using bolts with a pull-out capacity of 500 kN each. Whilst for second test rig (Figure 3.11), top column is tied back to the strong wall by a top column holder. Meanwhile, the bottom column is restrained by a steel column shoe, which was anchored to the strong floor. In addition, the column was bolted through inside the column shoe to ensure its rotation is minimized.
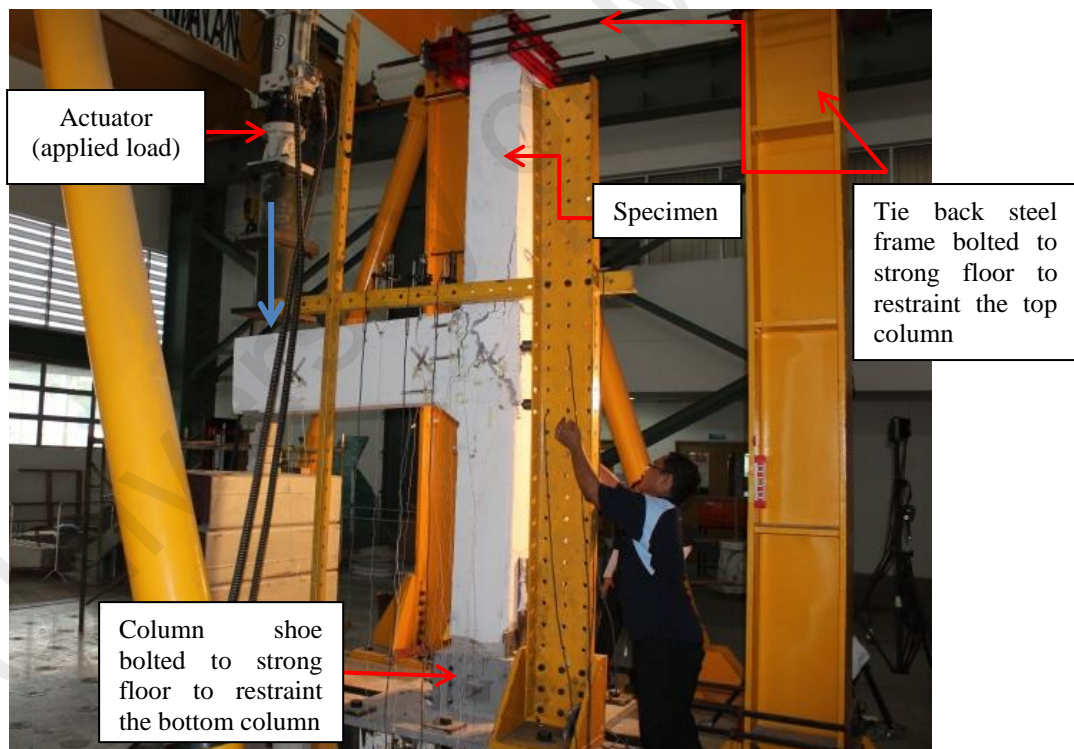
Figure 3.10: Experimental set up using tie back steel frame for experiment BHC1 and BHC2

The labels in the figure read:

Tie back steel and top column holder bolted to strong wall to restraint the top column

Strong wall

Actuator (applied load)

Specimen

Column shoe bolted to strong floor to restraint the bottom column
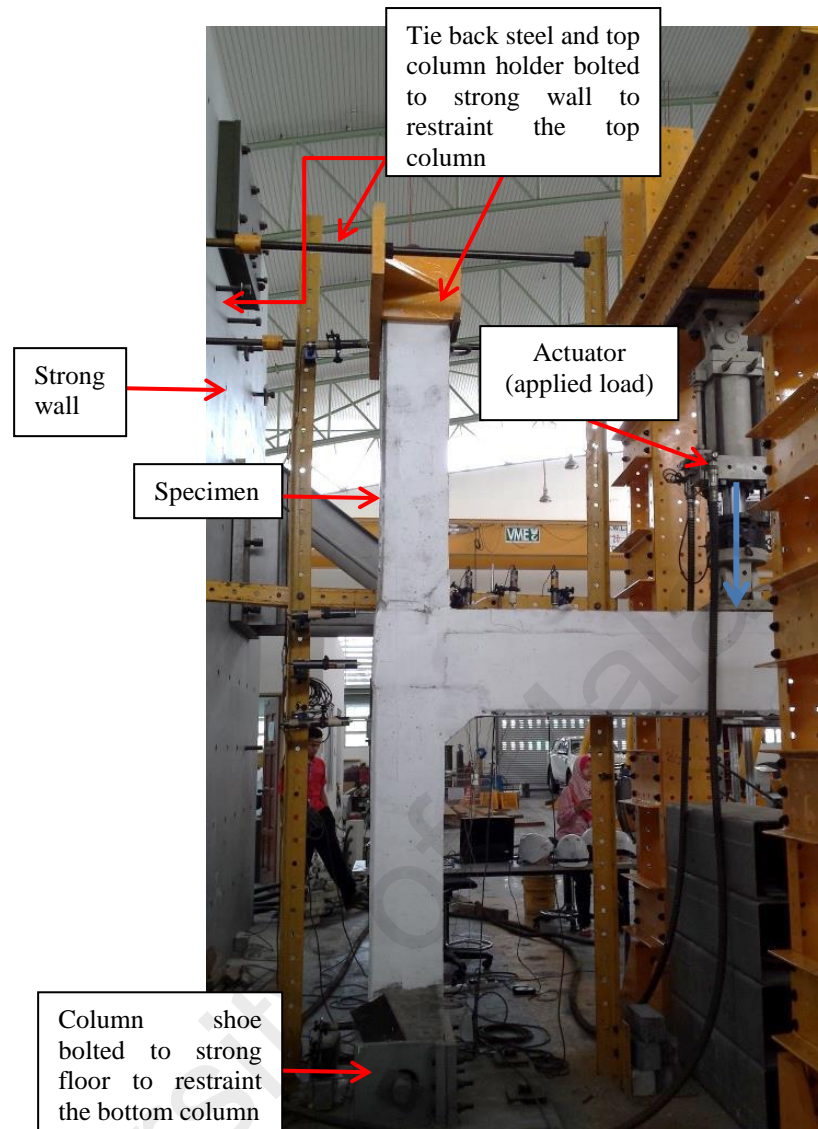
Figure 3.11: Experimental set up using strong wall for experiment BHC3

3.3.5   Instrumentation

Linear displacement transducers (LVDTs), concrete strain gauges, steel strain gauges, and load actuator were attached in several positions to monitor the connection's deformation, as depicted in Figures 3.12 – 3.13. All signals from sensors were automatically recorded and linked to a computer using a data logger. Moreover, respective calibration factors for the various sensors were input into the data logger to linearize signals. Then, logged data was transferred to and processed using Microsoft Excel 2010. Seven (7) numbers of LVDT were used to capture the displacement. The

distance between LVDT 2 to LVDT 4 was set up at 450 mm, while for LVDT 6 to LVDT 8 was located at 360 mm. Whilst, LVDT 1 and LVDT 5 were utilized to monitor the deflection at the top and bottom support during specimen tests.
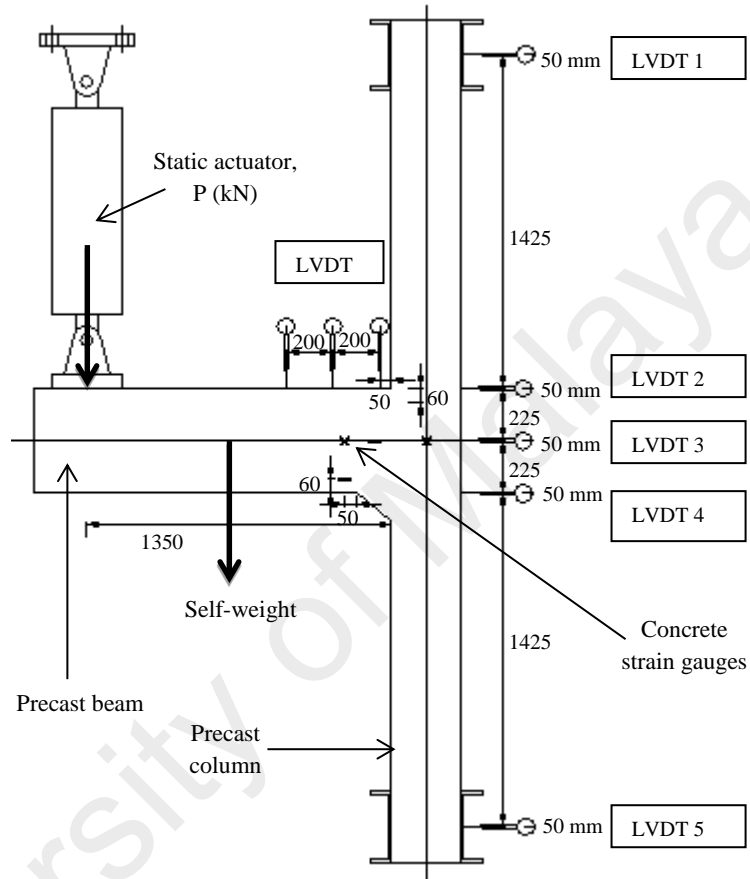
Figure 3.12: Schematic for instrumentation (LVDTs, actuator and concrete strain gauges)

(a) Position of concrete strain gauges

(b) Steel strain gauges location on tension steel cast inside concrete specimen (plan view of beam cross section)
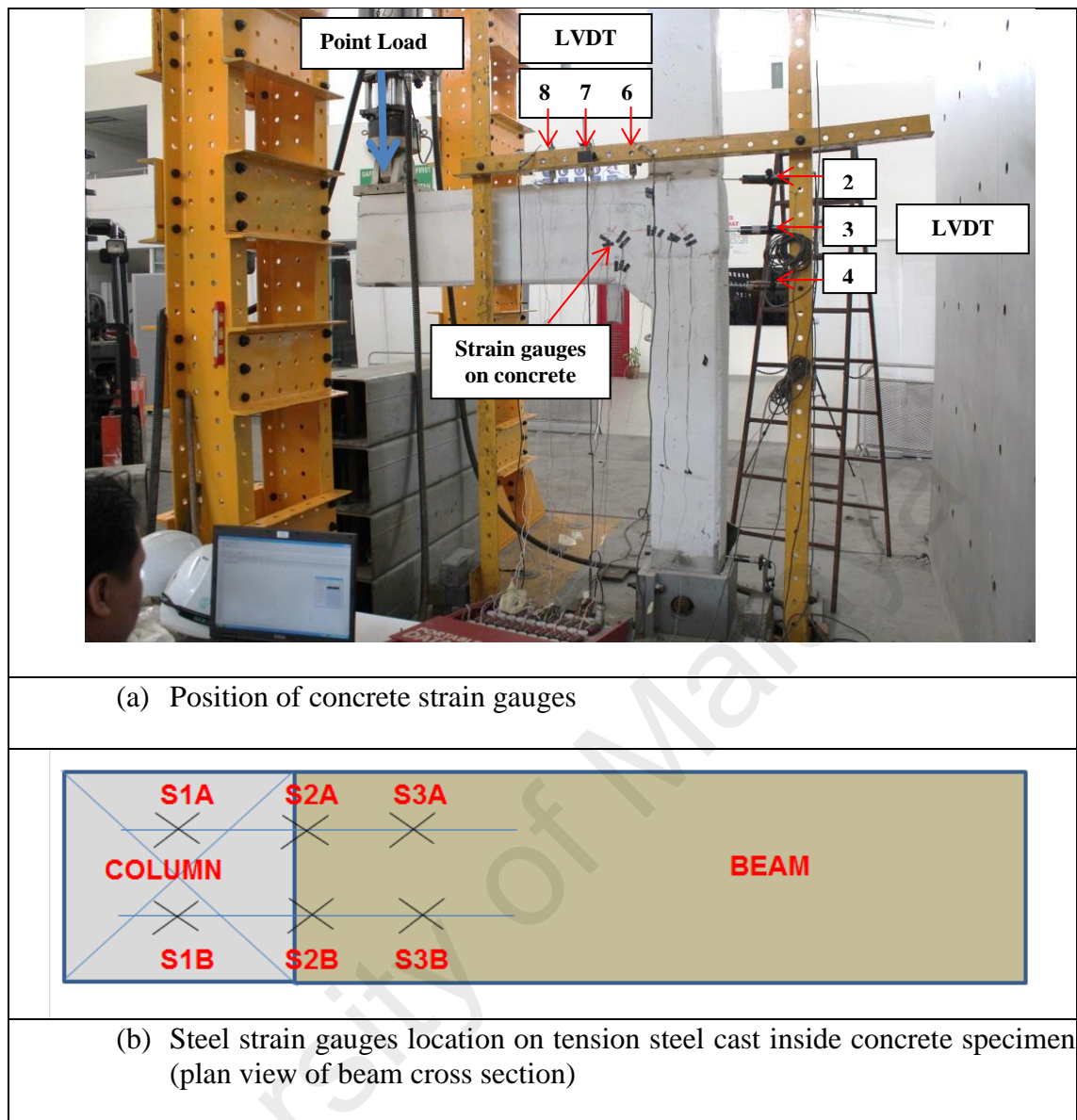
Figure 3.13: Placement of instrumentation (concrete and steel strain gauges)

3.3.6 Test Procedure

The test was conducted after the infill concrete strength for specimens had achieved 40 ± 5 N/mm². In order to study the stiffness of connections, the bending load, P was applied incrementally in six reversible cycles prior to monotonic loading. The load was reversed at first crack loading, second crack loading and then increased the load until the connections were not capable of supporting any further bending moment. Meanwhile, loading and unloading increments were 5 kN. This action gives the moment in the connector as M (kNm) include the selfweight of the beam. Besides that, relative rotation,

ϕ of the beam to the column were measured using a set of four linear displacement transducers (LVDT) at specific distances. Furthermore, readings of the gauges were recorded and cracks were marked at each load increments.

Figures 3.14 (a) – (b) show the important points and distances that involved in the calculation of moments and rotations. Hogging bending moment in the connection, M (kNm) was calculated by multiplying the magnitude of applied bending load, P (kN) by the lever arm of the beam, i.e. $M = Pa$. Furthermore, the distance between the line of action applied loads at the end of the beam and the face of the column is considered constant at 1.35 m. The connection rotation, ϕ was calculated for displacements, δ measured using four LVDTs at a specific distance and then divided by their respective distance (actual distance). Generally, moment and rotation are calculated as follows:

Moment, $M = 1.35P + 0.68\ selfweight$

$$= 1.35P + 0.68\ (3.24)$$

$$= (1.35P + 2.2032)kNm \tag{6}$$

Connection rotation, $\phi = \theta_{beam} - \theta_{column}$ \hfill (7)

where; $\qquad \theta_{beam} = (\delta_8 - \delta_6)/(x_8 - x_6)$

$\qquad\qquad \theta_{column} = (\delta_4 - \delta_2)/(x_4 - x_2)$

The final moments and rotations were then used in the presentation of the moment-rotation (M- ϕ) graphs.
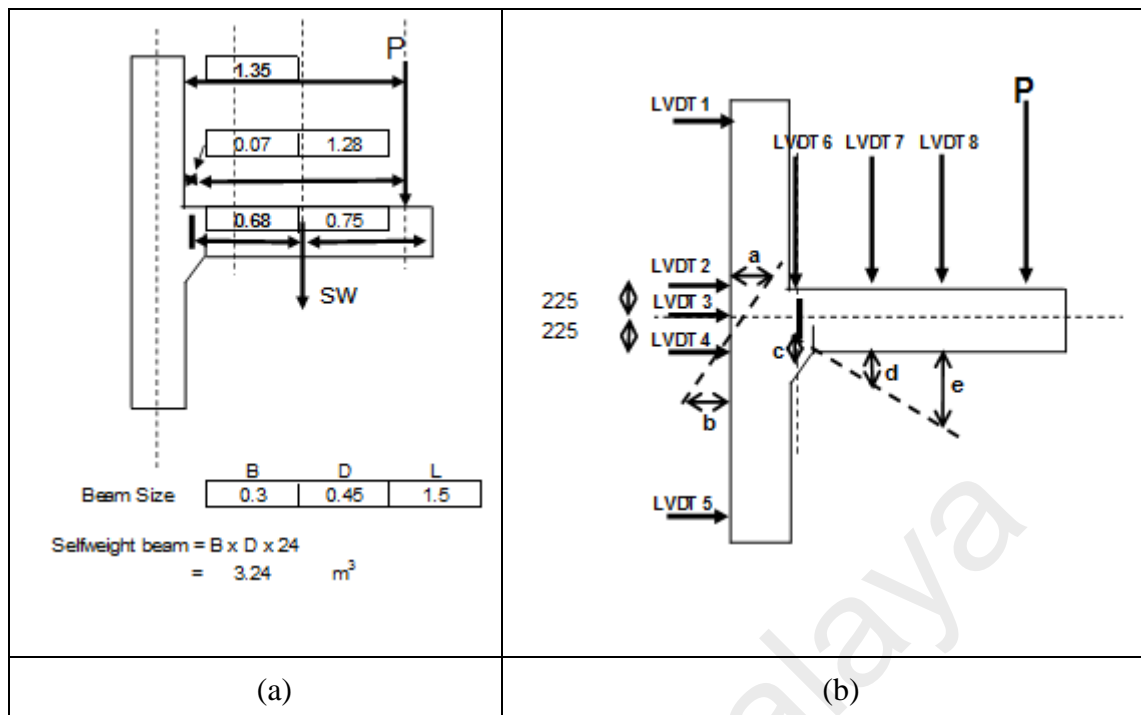
Figure 3.14: Moment rotation calculation

3.4 Method of Analysis

3.4.1 Calculation of Connector Moment Resistance, M_{RC}

The moment resistance of the connector, M_{RC} is calculated based on the equilibrium of all forces present in the connector, the beam reinforcement is excluded, but the tie steel above the beam is included as presented in Figure 3.15. The calculation model above is based on BS 8110 rectangular stress block using the as-tested material data (*in situ* infill concrete strength).

The equilibrium forces in connector $T = C$:

$$0.87f_y A_s = 0.45f_{cu}b\,0.9X$$

$$X = \frac{0.87f_y A_s}{0.45f_{cu}b\,0.9} \tag{8}$$

X is the depth of the stress block (mm), from the internal forces to be in equilibrium.

Then, moment resistance M_{RC}:

$$M_{RC} = Tz \tag{9}$$

The internal lever arm, z is the resultant of the horizontal forces. The detail example of calculation can be referred in Appendix E.
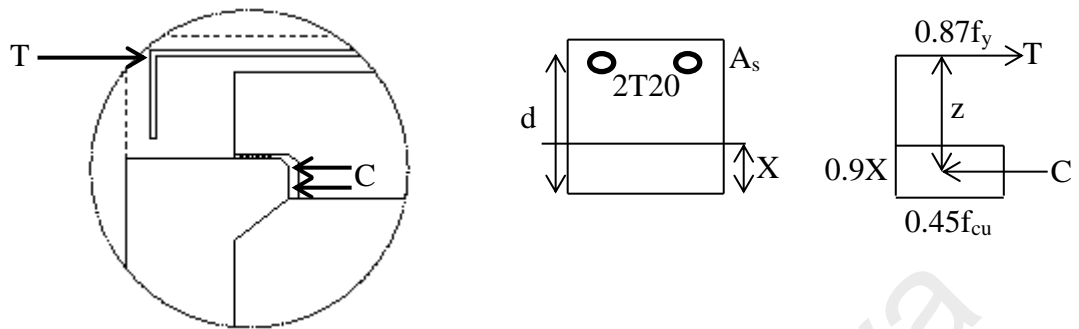
Figure 3.15: Calculation model of M_{RC}

3.4.2   Beam-line Method

From the investigation by Hasan *et al.* (2011), the beam-line method can be used in dealing with semi-rigid connections. Therefore, in this research, the beam-line method is used to quantify the relationship of M-ϕ and classification of connection. According to Elliott (2016) this beam-line approach is due to no connection is fully rigid or pinned in theory.
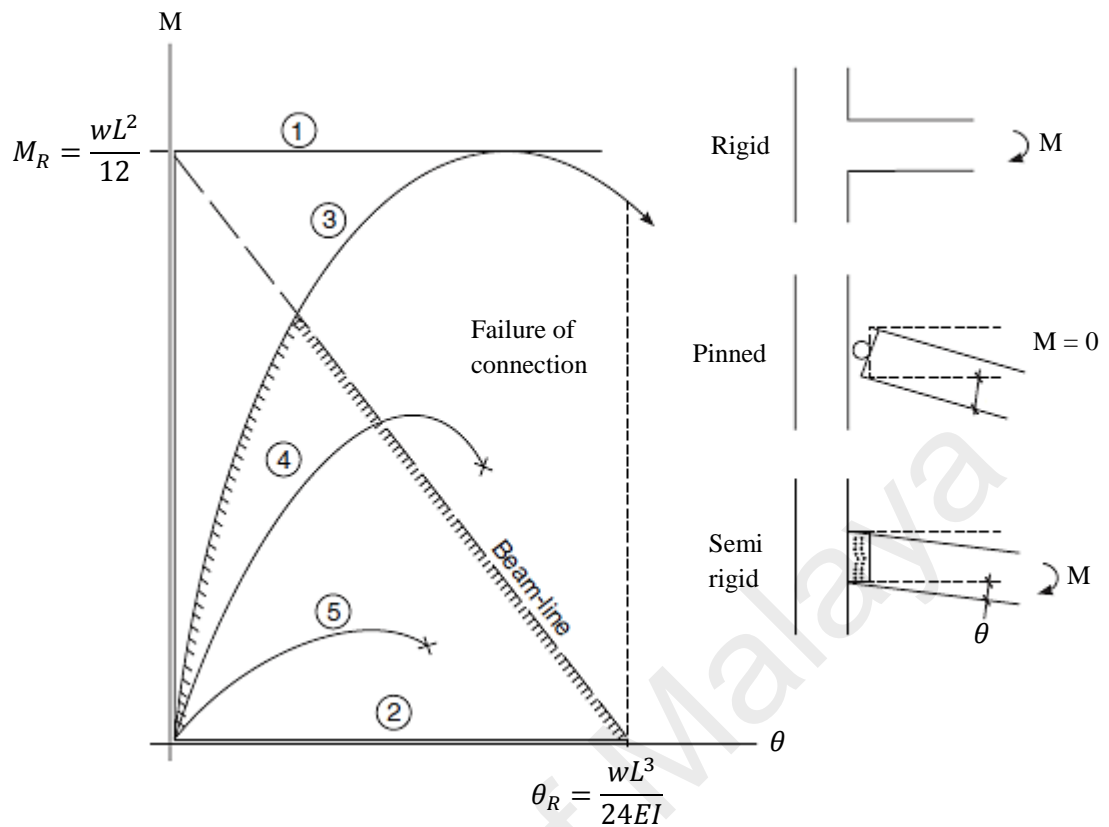
Figure 3.16: Definition of moment-rotation characteristics (Elliott, 2016)

Normally, all connections behave as a semi-rigid especially after the onset of flexural cracking. In Figure 3.16 the hogging moment of resistance of the beam at the support is given by $M_R > wL^2/12$ and the rotation of a pin-ended beam subjected to a UDL of w is $M = 0$ with $\phi = wL^3/24E_cI$ where E_c the short-term value for Young's Modulus and I is the flexurally cracked second moment area. The gradient of the beam line is $-2E_cI/L$. The plots 1 and 2 are the monolithic and pinned connections, respectively. However, in reality, the behaviour of a connection in precast concrete will tend to follow plots 3, 4 or 5 depending on the connection. If the plot follows plot 5, the connection should be considered as pinned.
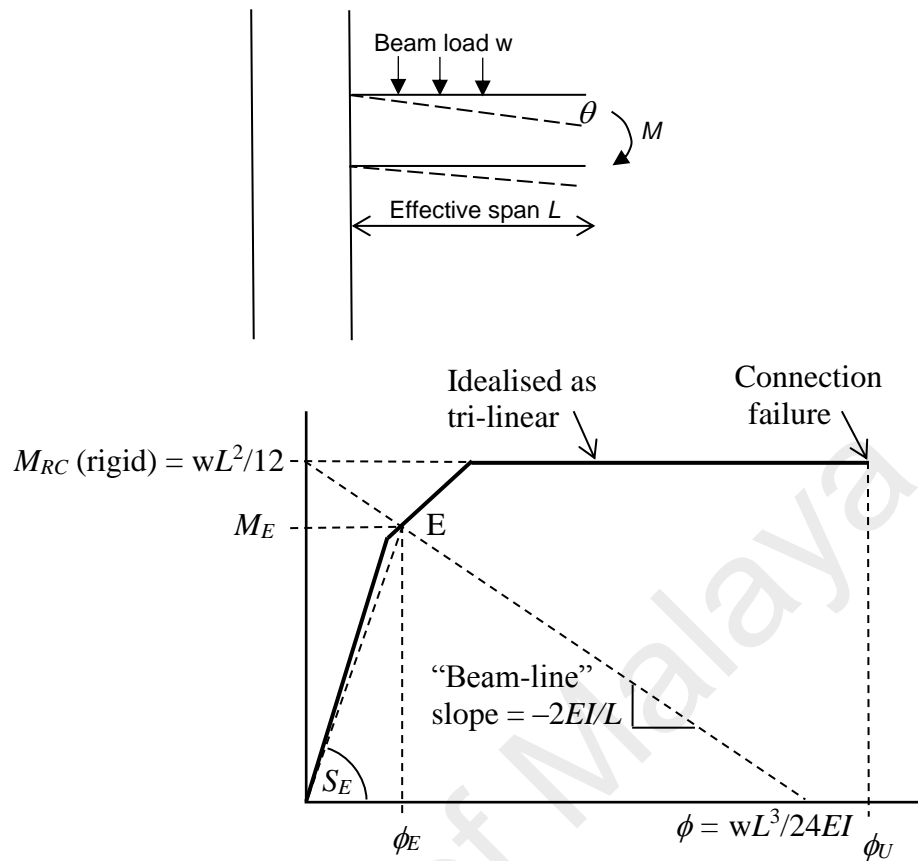
Figure 3.17 Definition of moment-rotation parameters for connections (Elliott and Jolly, 2013)

The relevant properties of semi-rigid connections are defined in Figure 3.17 as ultimate flexural strength M_R, rotational stiffness, S and ultimate rotational ductility capacity ϕ_u. M-ϕ behaviour in precast connections is typically linear up to about $0.3M_R$ and roughly parabolic afterward. The rotational stiffness is required at the point where the end moment and rotation in the connection match in other words beam-line intersect given by point E. All points along the beam-line define the M-ϕ relationship of the beam. The intersection point E defines the end moment and rotation of both the beam and the connection. If the connection fails prior to reaching point E, the connection is classified as pinned (even though there may be some strength and stiffness) (Elliott *et al.*, 2003).

3.4.3 Calculation of Beam-line Method and Stiffness (Elliott and Jolly, 2013)

The beam-line, of gradient $-2E_cI/L$ is drawn for the moment connection capacity, M_{RC} and for rotation ϕ_{RC} given by Eq. (10).

$$\phi = \frac{M_{RC}L}{2E_cI} \tag{10}$$

where E_c is taken as 30 kN/mm^2, and I is the second moment of area of the flexurally cracked beam. The beam span 6 m length is chosen as typical. Detailed calculations are presented in Appendix F. The intersection of M-ϕ plot with the beam-line gives the 'beam-End' requirement at point E where the moment, M_E and second stiffness, S_E are found. The stiffness ratio K_S of the connection is calculated for frame analysis purpose for future work. The stiffness S_E and stiffness ratio K_S was calculated from the slope of M-ϕ curve as follow:

$$S_E = \frac{M_E}{\phi_E} \quad , \qquad K_s = \frac{S_E}{4EI_c/L} \tag{11}$$

3.4.4 Fixity Factor

A classification system for precast concrete connections is shown in Figure 3.18 after Ferreira (Elliott *et al.*, 2005) wherein the semi-rigidity should meet both the strength and stiffness requirements simultaneously. Elliott and Jolly (2013) classified semi-rigid precast connections into three categories as partial strength, full strength and effectively rigid by setting conjugate limits for connection strength, stiffness and rotational capacity. To quantify the rotational stiffness, S of the connection, Monforton's fixity factor formula is adopted. This is a non-dimensional parameter that relates S to beam stiffness $3EI/L$. The formula for the varying value of $\gamma = 0$ for pinned to $\gamma = 1$ for fully rigid connections is given by Eq. (12).

$$\gamma = \left(1 + \left(\frac{3EI}{SL}\right)\right)^{-1} \tag{12}$$

The end moment M_E and the mid-span moment M_{span} are modified as a function of the fixity factor γ as follows

$$M_E = \frac{qL^2}{12}\left(\frac{3\gamma}{2+\gamma}\right) \quad and \quad M_{span} = \frac{qL^2}{12}\left(\frac{3-1.5\gamma}{2+\gamma}\right) \tag{13}$$

Referring to Figure 3.18, Ferreira identified two important demarcation points:

- $\gamma = 0.4$ distinguishes low strength from medium strength, because when $\gamma > 0.4$ the semi-rigid behaviour provides more than 50 per cent of full rigidity

- $\gamma = 0.67$ distinguishes medium strength and high strength because $M_E > M_{span}$

Therefore, the proposed classification system after Ferreira (Elliott *et al.*, 2005) consists of five distinct zones as indicated in Figure 3.18.

1) ZONE I (pinned connections): $\gamma \leq 0.14$. There is no semi-rigid behaviour, but connections must provide adequate rotation capacity in order to guarantee integrity when the beam end rotates.

2) ZONE II (semi-rigid with low strength): $0.14 < \gamma \leq 0.40$. Connections are not able to act as moment-resisting.

3) ZONE III (semi-rigid with medium strength): $0.40 < \gamma \leq 0.67$. Connections are suitable to act as moment-resisting connections but still cannot provide semi-continuity to the adjoining beams.

4) ZONE IV (semi-rigid with high strength): $0.67 < \gamma \leq 0.90$. Connections are fully connected and provide semi-continuity to the adjoining beams.

5) ZONE V (rigid connections): $\gamma > 0.90$. There is no need to consider the semi-rigid behaviour. However, even rigid connections should be assessed in terms of ductility.
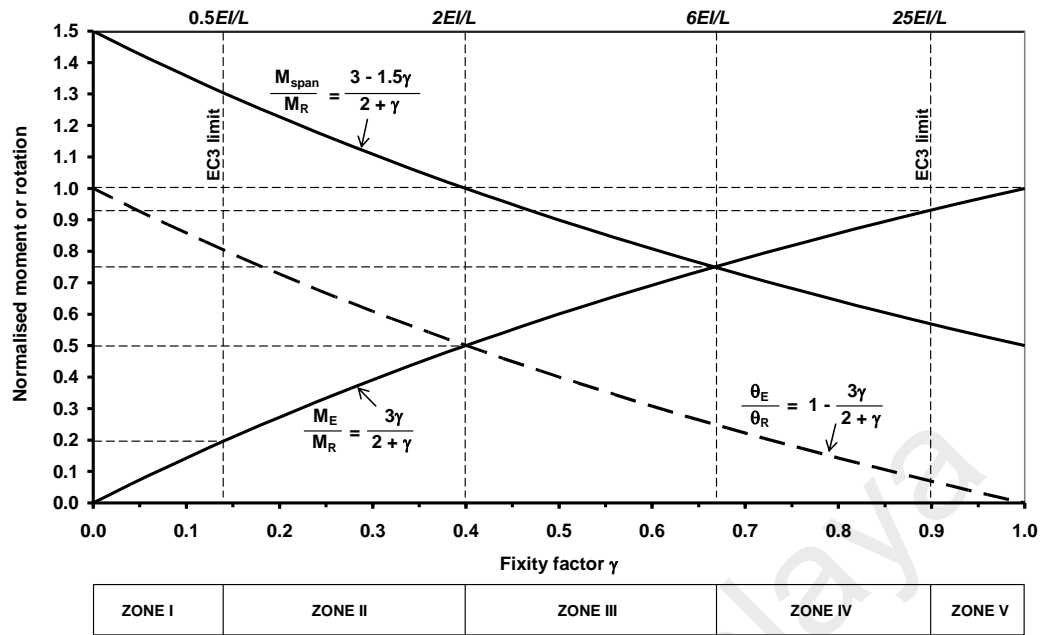
Figure 3.18: Classification system for pinned, semi-rigid, fully rigid beam-to-column connections after Ferriera (Elliott *et al.*, 2005 and Elliott & Jolly, 2013).

CHAPTER 4: RESULTS AND DISCUSSIONS

4.1 Introduction

Beam-to-column connections are crucial for structural in developing frame action in precast concrete buildings. The connection must have sufficient strength to resist the applied loads and adequate stiffness to limit sideways movement of the overall structure. This chapter presents all the results obtained from three connection tests accordingly in the form of a graph. The results are presented from derived calculations including hogging bending moment (M-ϕ) plot, load-displacement, load-steel strain and failure mechanism. Besides, the graphical outputs of moment-rotation and mode of failure are assembled in order to assist comparison and discussion of the performance of the connections.

4.2 Connector Moment Resistance, M_{RC}

Predicted moment and experimental values are tabulated in Table 4.1. Moment resistance (M_{RC}) value for BHC1, BHC2 and BHC3 is 111.58 kNm, 122.72 kNm and 124.13 kNm, respectively. As shown in Table 4.1, maximum moment of connections, M_U, was generally less than calculated moment resistance (M_{RC}). M_{RC} is based on all structural components present at the column face achieving their full yield capacity. Internal level arm z is resultant of horizontal forces of the lapping tension bars. This calculation is without partial safety factors for precast concrete and steel rebars. Furthermore, the compressive stress in the concrete infill is equal to $0.67f_{cui}$. Based on these assumptions, value for BHC1 connection M_U / M_{RC} is 0.98 while for BHC2 it is 0.86 and for BHC3 is 0.99.

Table 4.1: Results of negative bending tests

Connection	Material properties data from experiments (mean value)		Predicted values	Test values		
	Infill cube strength, f_{cui} (N/mm^2)	Bars yield strength, f_y (N/mm^2)	Connector moment capacity, M_{RC} (kNm)	Moment at first crack, M_{CR} (kNm)	Connector test ultimate, M_U (kNm)	Ratio of M_U/M_{RC}
BHC1 (T1)	41.0	539.5	111.58	42.7	109.77	0.98
BHC2 (T2)	46.5	592.0	122.72	42.7	105.30	0.86
BHC3 (T3)	56.0	592.0	124.13	42.7	123.62	0.99

4.3 Moment-rotation Graph

The derived moment-rotation graph gained from beam-line calculation method described in Sections 3.4.2 – 3.4.3 is presented in Figures 4.1 – 4.3. The graph constitutes loading and unloading data for BHC1, BH2 and BHC3 connections with ϕ_{RC} of 19.41 m.rad, 21.38 m.rad and 21.40 m.rad, respectively. The graphs were plotted using data in Appendices G – I. Then, from the plotted graph, M_E is 54 kNm and ϕ_E is 10.0 m.rad; therefore, the value of S_E is 5.40 kNm/m.rad. Moreover, the ratio of connector-to-beam stiffness K_S is 0.47. Corresponding values for BHC2 and BHC3 connections are given in Table 4.2.

Table 4.2: Experimental test moments, rotations and stiffness

Connection	Beam-end rotation at E, ϕ_E (m.rad)	Beam-end moment at E, M_E (kNm)	Beam-end second stiffness at E, S_E (kNm/m.rad)	Ratio of K_S at E
BHC1 (T1)	10.0	54	5.40	0.47
BHC2 (T2)	9.1	70	7.69	0.67
BHC2 (T3)	8.3	75	9.04	0.78

After forces of the actuator were applied incrementally on the beam, rotation increased steadily. Ultimately, corbel connection (ϕ_u) attained maximum values of 38.2 m.rad, 23.8 m.rad and 56.0 m.rad at the maximum moment (M_U) of 109.8 kNm, 105.3 kNm and 123.6 kNm, respectively. Despite this, the test was stopped as soon as excessive cracks and spalling of concrete occurred in the connection zone.

BHC1 and BHC2 specimen's stiffness line is observed to be bi-linear, which is marked at Points B and C in Figures 4.1 – 4.2. The B-C line is its new stiffness after initial cracking. Consequently, B-C line has the same gradient with D line (cycle of the second crack) and shows little loss of strain energy after initial cracking.

Clearly, specimen BHC3 (Figure 4.3) showed increasingly increment of rotations for each cycle. This might happen due to the damage at the bottom of the column when setting up the specimen before testing.
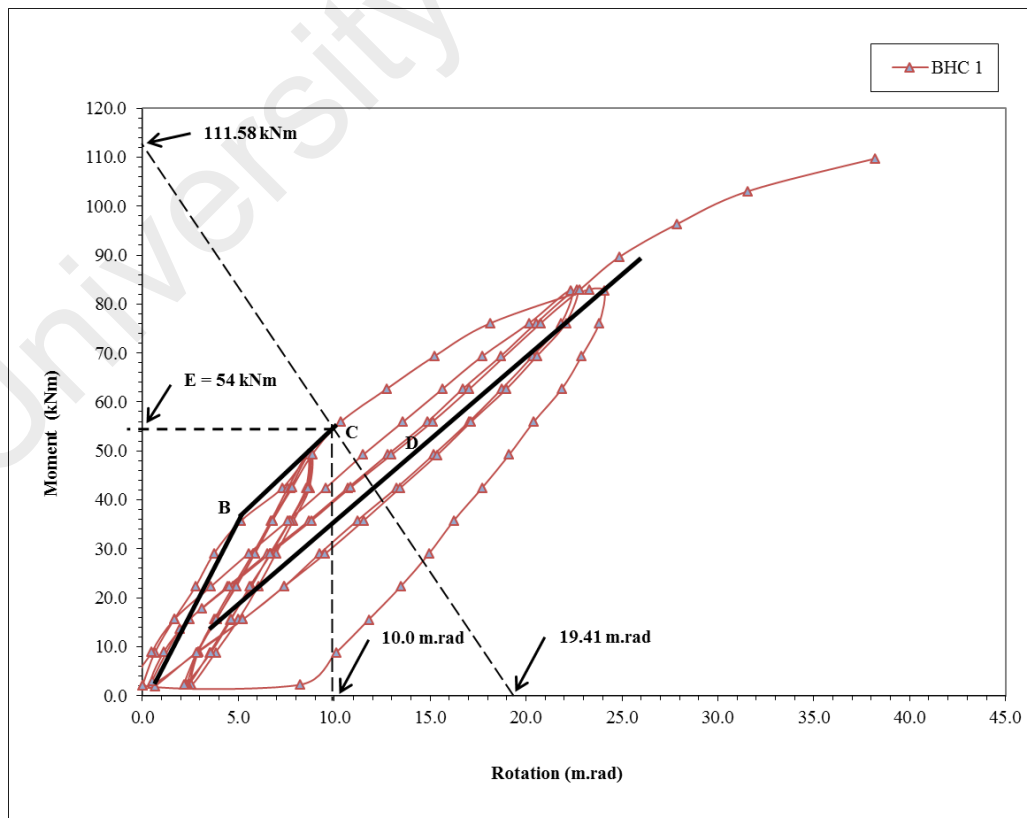
Figure 4.1: Beam-line plot for BHC1
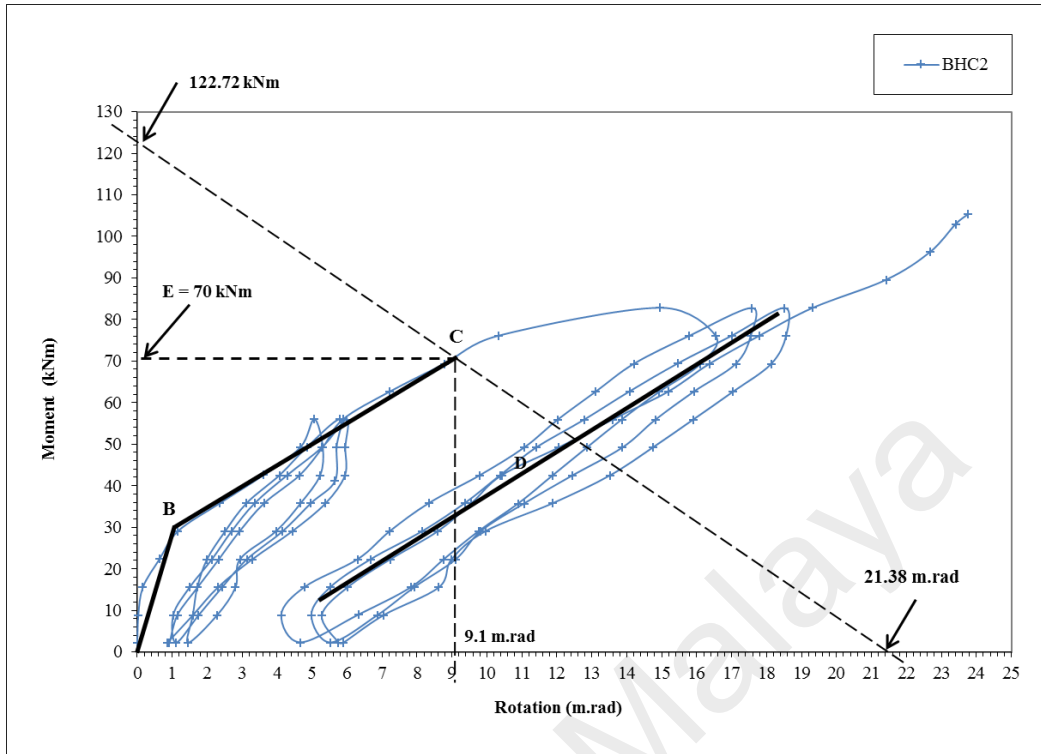
Figure 4.2: Beam-line plot for BHC2
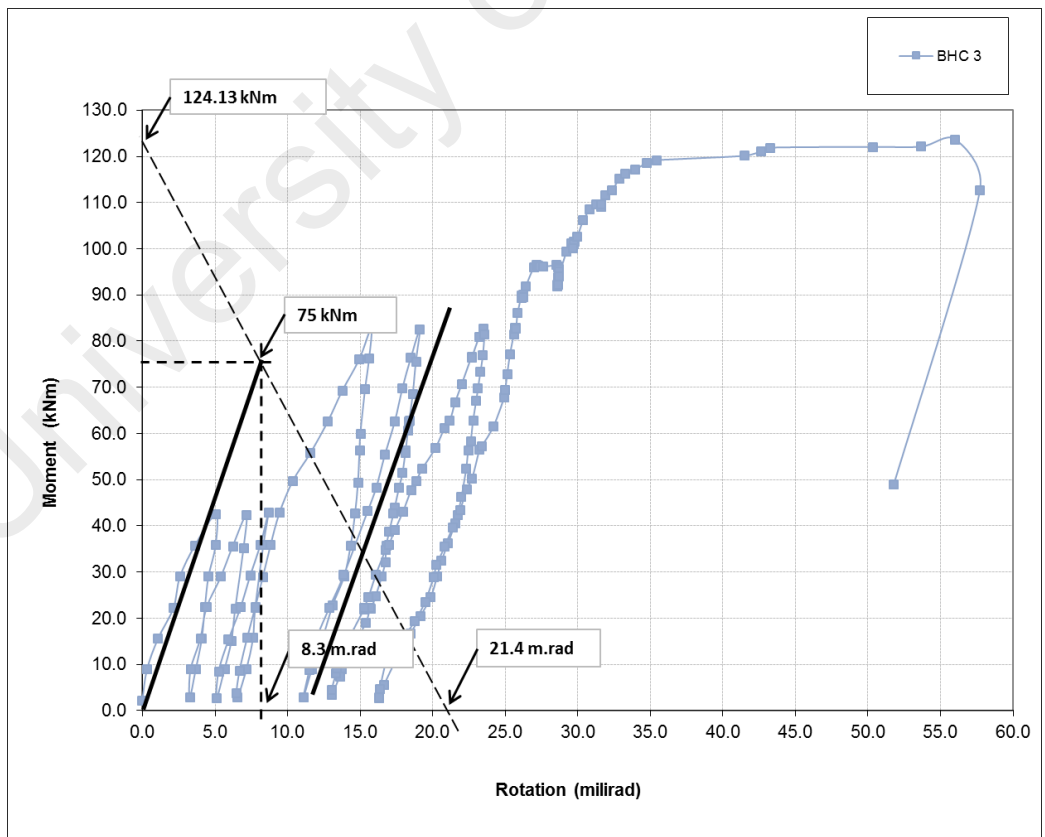
Figure 4.3: Beam-line plot for BHC3

4.4 Relationship between Moments and Rotations in the Beam and Connector

Based on calculated values and plotted graphs (Figure 4.4), beam-line of both BHC2 and BHC3 are almost similar and curve pattern shows that these proposed precast connections can be classified as semi-rigid. Meanwhile, BHC1 had lesser stiffness compared to other connections.
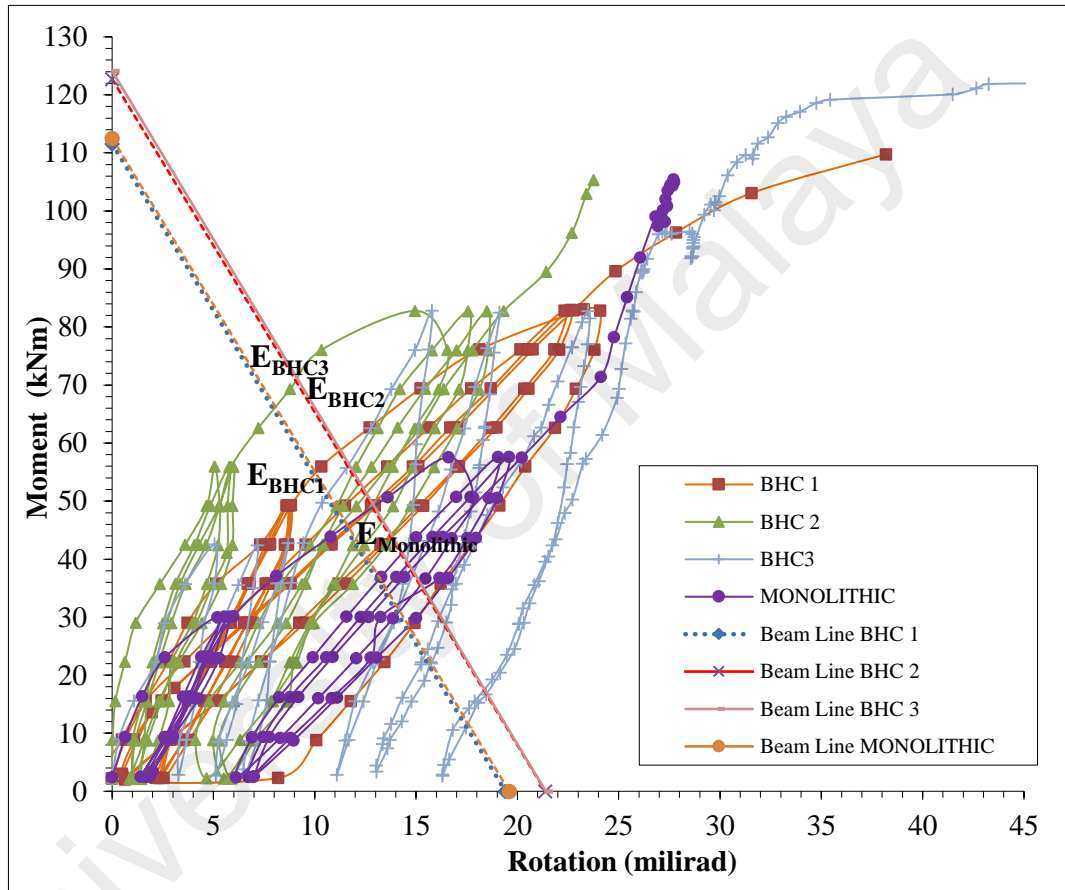
Figure 4.4: Moment-rotation curves for all specimens

Besides that, the monolithic specimen graph is drawn to provide a comparison of pattern trends between precast and monolithic connections. This monolithic specimen was cast in a conventional manner. The detail of main reinforcements for column and beam in monolithic are the same as proposed connections. In addition, reinforcement volume ratio in precast components is duplicated in the connection zone due to lapping and precast half beam reinforcement arrangement. This extra reinforcement is the reason

for higher stiffness in precast connections. According to Wahjudi *et al.* (2014), Breccolotti *et al.* (2016), and Guan *et al.* (2016), strength and ductility of precast and cast-in-situ joints are very similar. Moreover, these joints are more resistant and stiffer than a monolithic joint without any appreciable changes to the ductility of the joint.

4.5 Classification of Connections

A classification system, developed by Ferreira, for precast concrete connections as shown in Figure 3.18, wherein semi-rigidity should meet both strength and stiffness requirements simultaneously. By using fixity factor formula, I used as flexurally cracked the second moment of area of the beam and S_E gives γ, 0.47 for BHC2 and γ, 0.51 for BHC3 meanwhile 0.39 for BHC1 as shown in Table 4.5. According to Figure 3.18, precast beam-to-column connections BHC1 can be classified as semi-rigid with zone II (semi-rigid with low strength); BHC2 and BHC3 with zone III (semi-rigid with medium strength), where fixity factor γ is between 0.40 and 0.67. Connection in zone III is suitable to act as moment-resisting, but it still cannot provide semi-continuity to adjoining beams. Furthermore, in this region, taking $\gamma = 0.47$, beam M_{span} is still about 38% greater than M_E. In addition, for uniformly distributed loading w, end moment $M_E = wL^2/21.1$, compared with $-wL^2/12$ for encastre. Meanwhile, $M_{span} = +wL^2/12.9$, which is evidently an improvement over $+wL^2/8$.

4.6 Load-displacement Relationship

Figures (4.5) – (4.10) represent the load-displacement in the beam-to-column joint for BHC1, BHC2 and BHC3. The displacements were recorded at every 5 kN increment. After loads were constantly applied, the deflection increased progressively for every stage and cycle. Moreover, prior to the appearance of the second cracks, connections were deflected elastically. Thus, the load-displacement relationship was nearly linear.

Nevertheless, after the appearance of the second cracks, the load-displacement relationship became nonlinear and reached a maximum value.
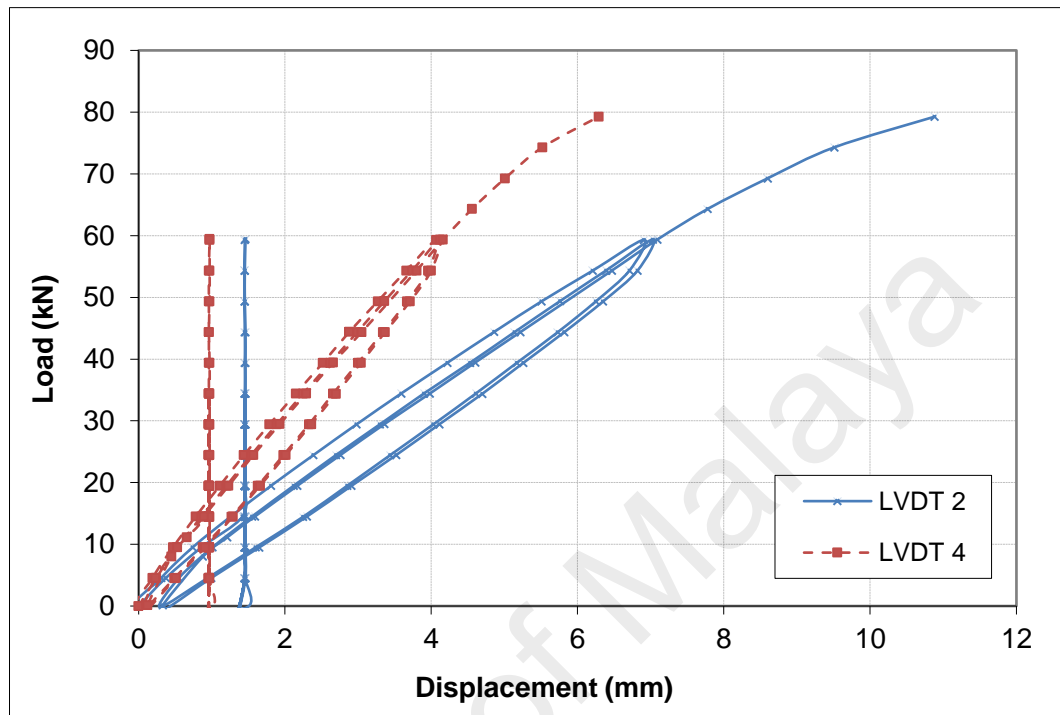
Figure 4.5: Load-displacement of BHC1 for LVDT (2) and (4)

The load resistance of specimen BHC1 was achieved at the maximum value of 79.68 kN with maximum displacements recorded at precast column 10.88 mm (Point 2) and 6.29 mm (Point 4) (Figure 4.5). Whereas, the precast beam had deflected 6.52 mm and 11.52 mm at Points (7) – (8) respectively as shown in Figure 4.6.
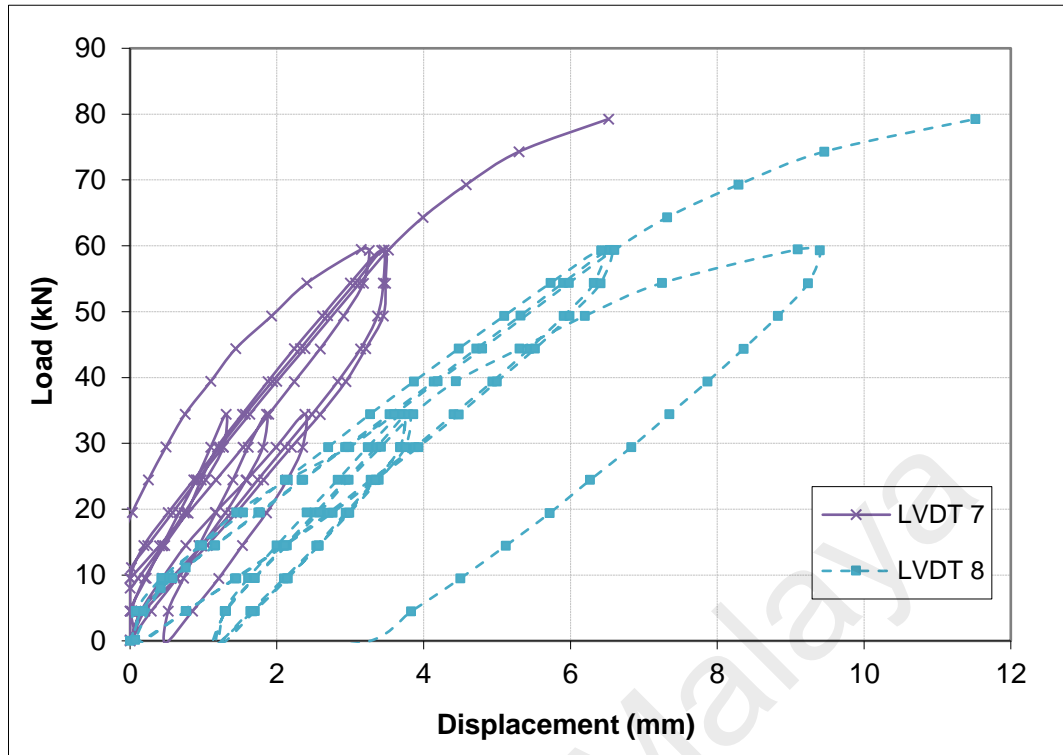
Figure 4.6: Load-displacement of BHC1 for LVDT (7) - (8)

Meanwhile, the load-displacement graph for BHC2 is illustrated in Figures (4.7) – (4.8). The load resistance of specimen BHC2 was attained at a maximum value of 76.38 kN with maximum displacements recorded at precast column at 16.51 mm (Point 2) and 14.44 mm (Point 4). While the precast beam had deflected at 10.08 mm and 8.99 mm at Points (7) – (8) respectively.
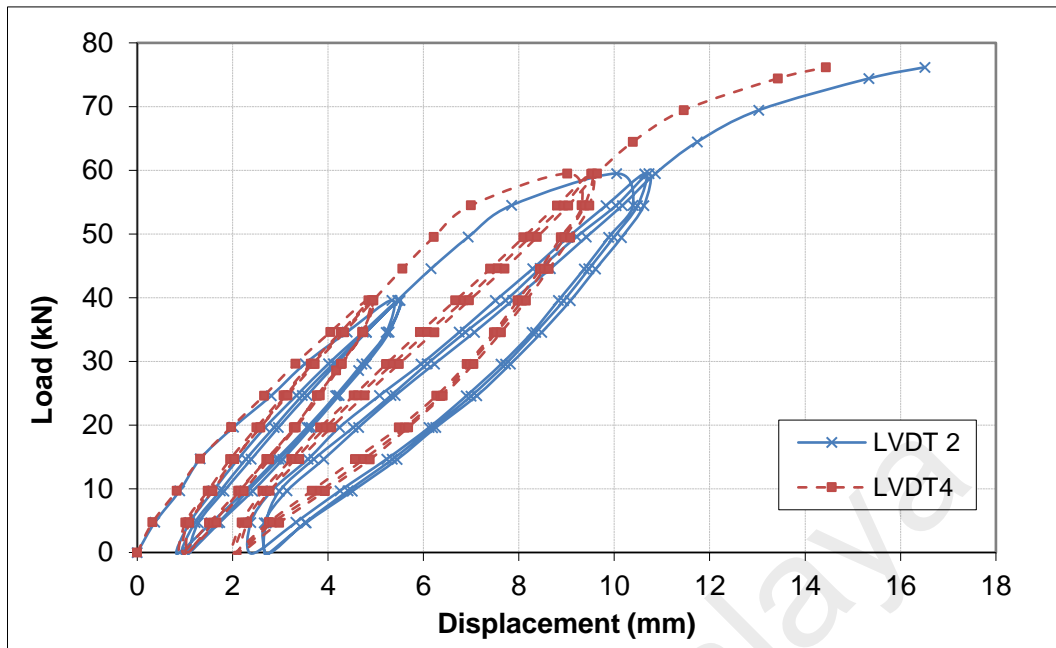
Figure 4.7: Load-displacement of BHC2 for LVDT (2) and (4)
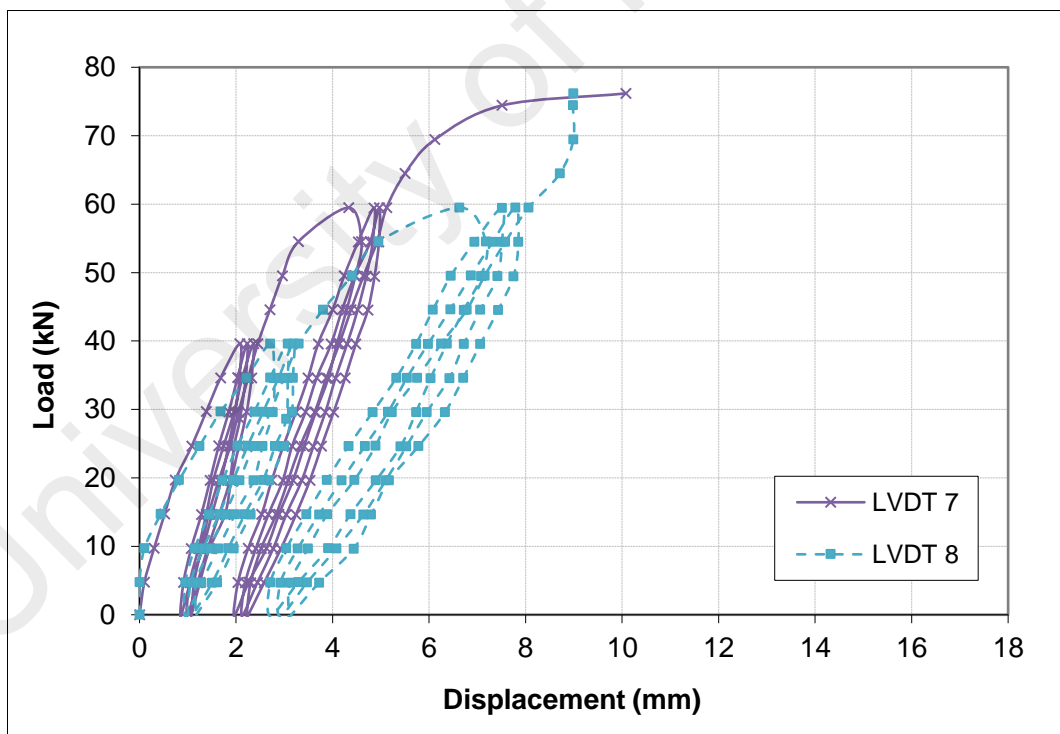
Figure 4.8: Load-displacement of BHC2 for LVDT (7) - (8)

The load-displacement curves in connection for BHC3 are presented in Figures (4.9) – (4.10). The load resistance of specimen BHC3 was attained at a maximum value of 89.94 kN with maximum displacements recorded at precast column 18.33 mm (Point 2) and 12.97 mm (Point 4). The readings for Points (7) – (8) were recorded as 8.83 mm and 18.66 mm, respectively. The results show the displacements induce the rotation between the beam and column for all connections.
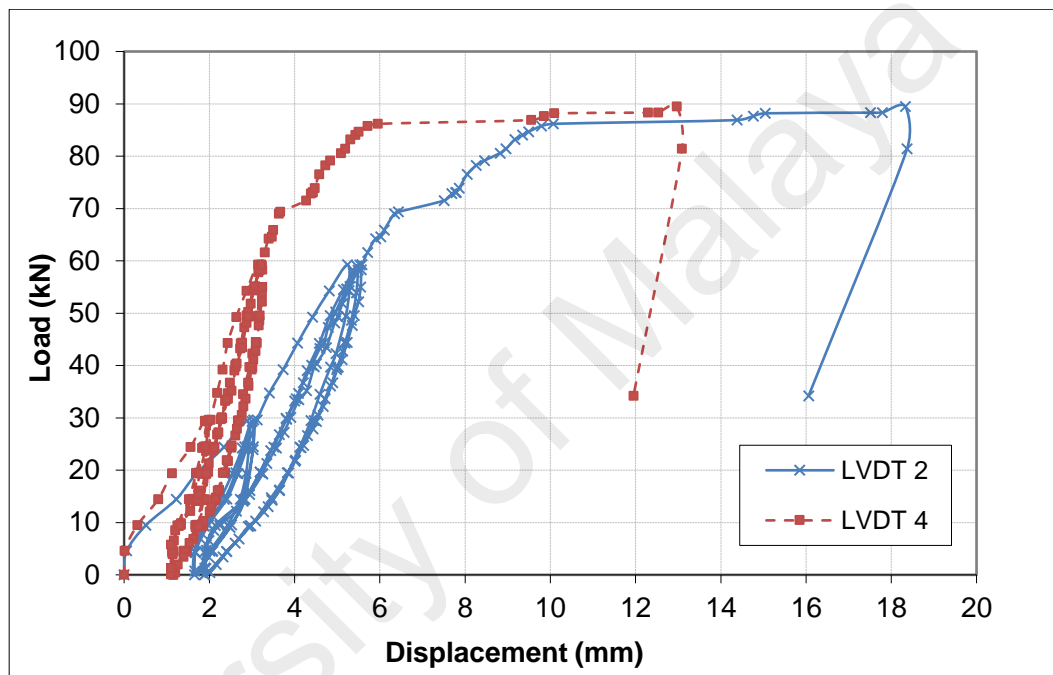
Figure 4.9: Load-displacement of BHC3 for LVDT (2) and (4)
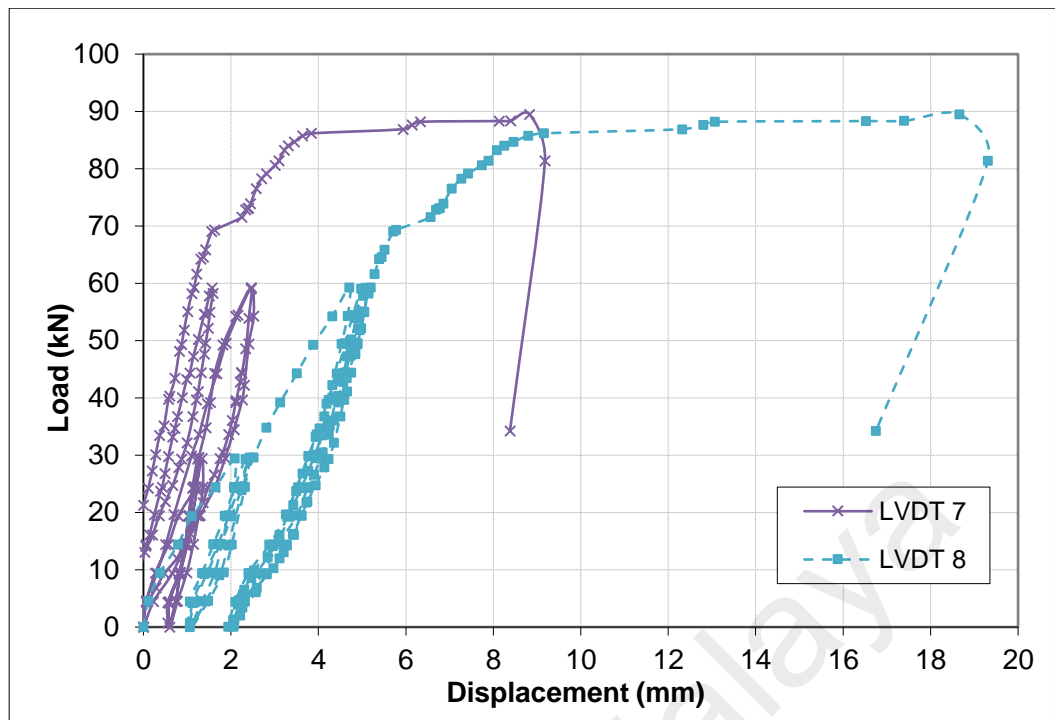
Figure 4.10: Load-displacement of BHC3 for LVDT (7) - (8)

Comparison between three specimens in which all specimens show trends of curves in a similar manner is illustrated in Figure 4.11. Hence, connections are in a ductile manner. Besides that, ductility factor is defined by the ratio of ultimate displacement to displacement at yielding of tensile reinforcement (Farnoud *et al.*, 2015). Meanwhile, the over-strength factor is the ratio between ultimate load and load at first yield (Wahjudi *et al.*, 2014). The displacement ductility factor for BHC1 of 1.63 and BHC2 of 1.97 gives a 21% improvement in ductility. This can be attributed to increasing anchorage length in the connection zone. The ductility factor for BHC3 is 1.70. The over-strength factors for BHC1, BHC2 and BHC3 are 1.23, 1.28 and 1.51, respectively, as given in Table 4.3. All the data of load and displacement for each connection are provided in Appendix (J) – (L).
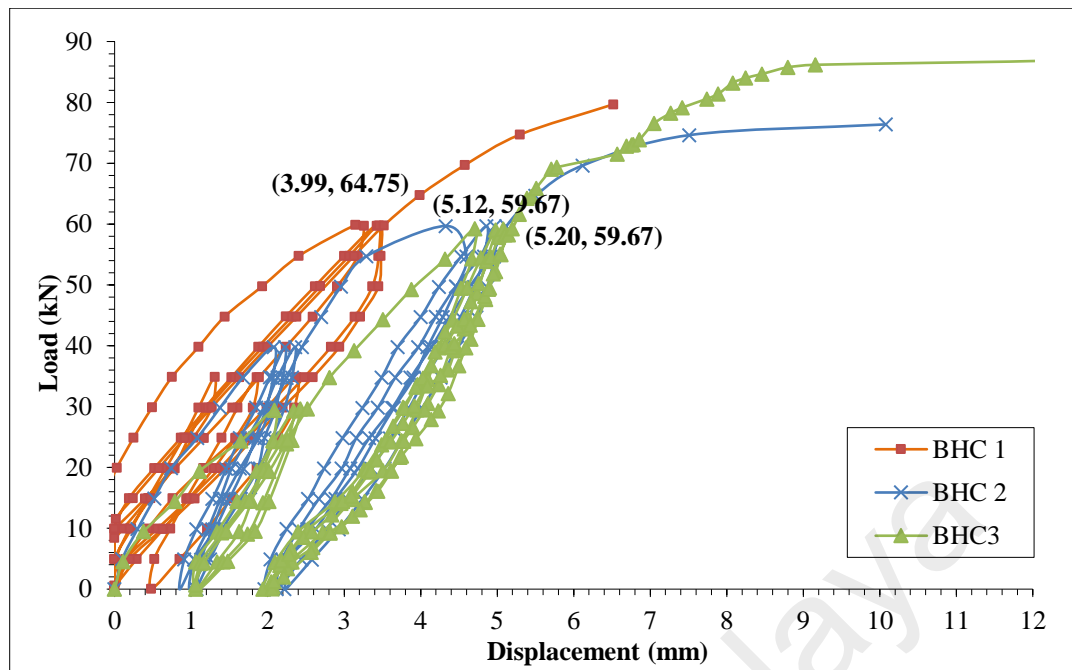
Figure 4.11: Load-displacement graph (BHC1, BHC2 and BHC3)

Table 4.3: Ductility and over-strength factor

Connection	Yield load (kN)	Ultimate load (kN)	Displacement at yield load	Displacement at ultimate load	Ductility factor	Over-strength factor
BHC1 (T1)	64.75	79.68	3.99	6.52	1.63	1.23
BHC2 (T2)	59.67	76.38	5.12	10.08	1.97	1.28
BHC3 (T2)	59.67	89.94	5.20	8.83	1.70	1.51

4.7   Load-steel strain Relationship

Displacements are a measurement of elastic and plastic deformations of connections and as a whole represents a concrete strain release in tension while increasing steel strains at the cracked section. Load versus steel strains are shown in Figure 4.12. Strain gauges were installed on the tension bar before casting infill concrete. The load was applied gradually and strain in between column and beam was measured at every cycle of loading. Limitation of strains is defined in joint. The strain of top longitudinal bar in BHC1, BHC2 and BHC3 did not yield at all, which had attained a maximum rate of 1697 µε, 1682 µε and 1675 µε, respectively, before failure. In general, strains of tie bars did not reach their uniaxial yield; indicating that full plastic moment for connections was not achieved. This can be explained as a failure of connections due to extensive flexural cracking in connection zone that led to bond-slip occurring in top bars.
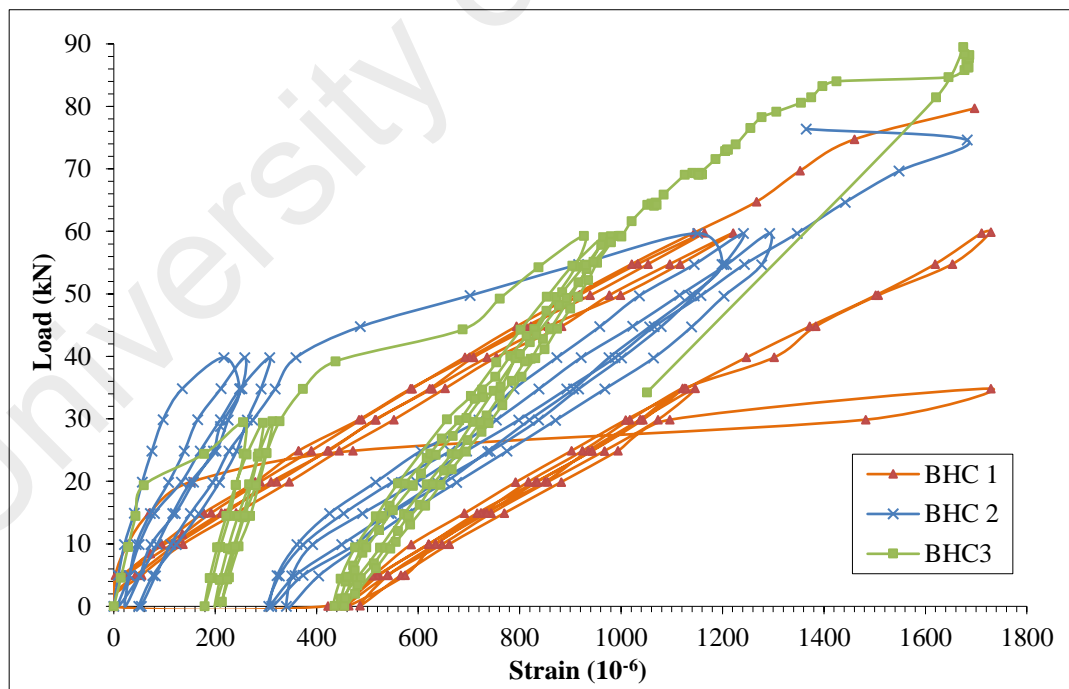
Figure 4.12: Load versus steel strain for BHC1, BHC2 and BHC3

4.8 Crack Pattern and Failure Mode

Figures 4.13 – 4.16 show bending and shear cracks due to negative bending moment and shear force in concrete corbel. Cracks in corbel developed in a more gradual manner, whereby as shown in earlier moment-rotation plots, there was dissipation energy prior to ultimate failure. In both BHC1 and BHC2, the first flexural crack appeared in in-situ concrete infill just inside the column and occurred at P = 30 kN or M = 42.7 kNm. Furthermore, this led the bar to be subjected to an eccentric tie force; thus, reducing axial stiffness. This value is in good agreement with the theoretical cracked value (40 kNm), based on a flexural tensile stress of 2.52 N/mm^2 (0.37 $(f_{ct})^{1/2}$). The test revealed that as expected, cracks were initiated at the column to precast beam joint interface. This occurred due to relative strength and stiffness weakness of the joint's two different materials. Moreover, discontinuity of concrete in the beam-column interface has resulted in a less rigid connection. Different to BHC1 and BHC2, the first crack of the BHC3 connection happened at 25 kN of applied load or M = 35.8 kNm. It was seen that it happens from the face of spalling precast half column to the precast half beam.

For the second stage, it was observed that cracks grew and widened in a diagonal pattern at M = 83.2 kNm. Finally, these cracks caused splitting along the face of the beam-column intersection and diagonally at the connection, until it failed at ultimate moment of M_U = 109.77 kNm, 105.3 kNm and 123.62 kNm, respectively. In addition, flexural cracks for both connections penetrated to the infill concrete region, which indicates a good integrity between beam and column components in precast connections. Moreover, shear cracks were also observed along the precast column.

At failure point, slippage failure was observed in BHC1, which indicates that anchorage failure occurred in connection zone. Design Guidelines from Architectural Institute of Japan (AIJ) divided anchorage failure in exterior beam-column joints into

three modes, namely, side split failure, local compression failure, and raking-out failure (AIJ, 1999). As seen in Figure 4.14, the failure mode of BHC1 is therefore categorised simultaneously under side split failure of concrete beside column and raking-out failure. These types of failure are due to split stress around the inside of the bend portion of bars (Joh and Goto, 2000).
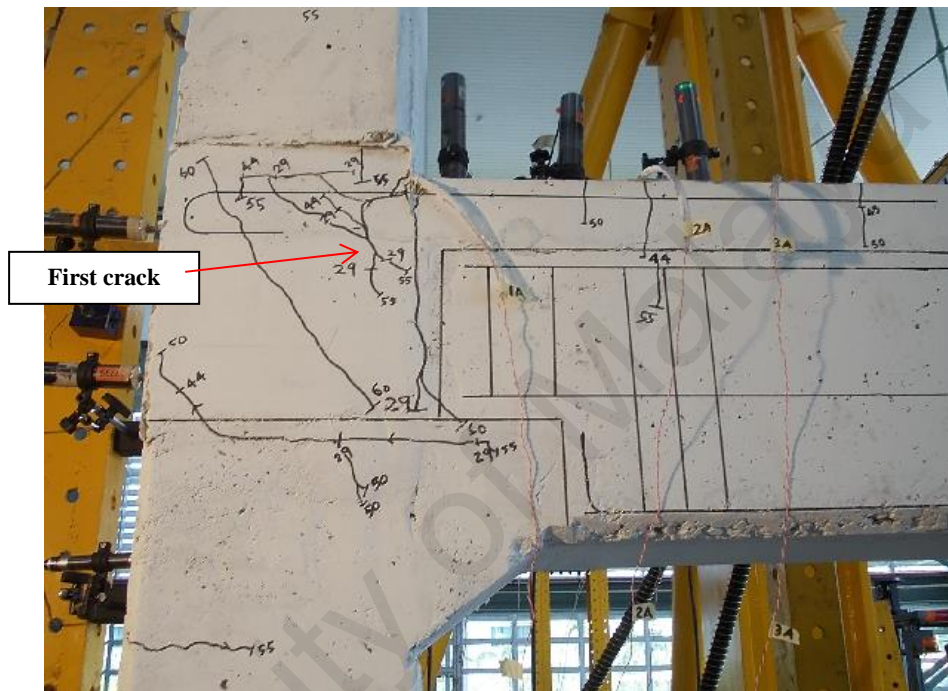
Figure 4.13: Second stage cracks for BHC1
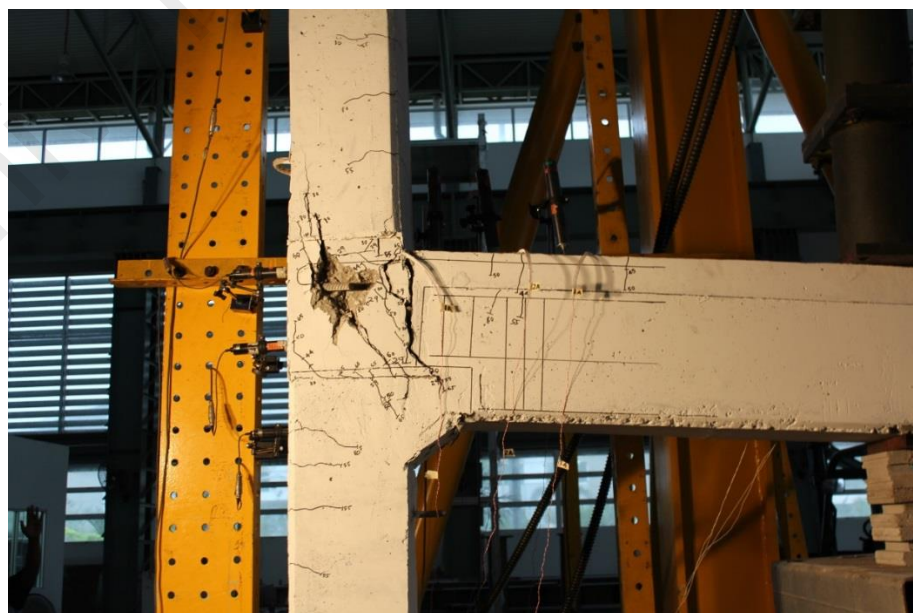
Figure 4.14: Failure mode for BHC1

Meanwhile, for BHC2 and BHC3, bond-slip accrued in reinforcement bars due to the splitting of concrete that develops along the interface and extensive diagonal cracking in connection zone. In addition, Figure 4.15 and 4.16 show that BHC2 and BHC3 column had a spalling effect when the connection failed.
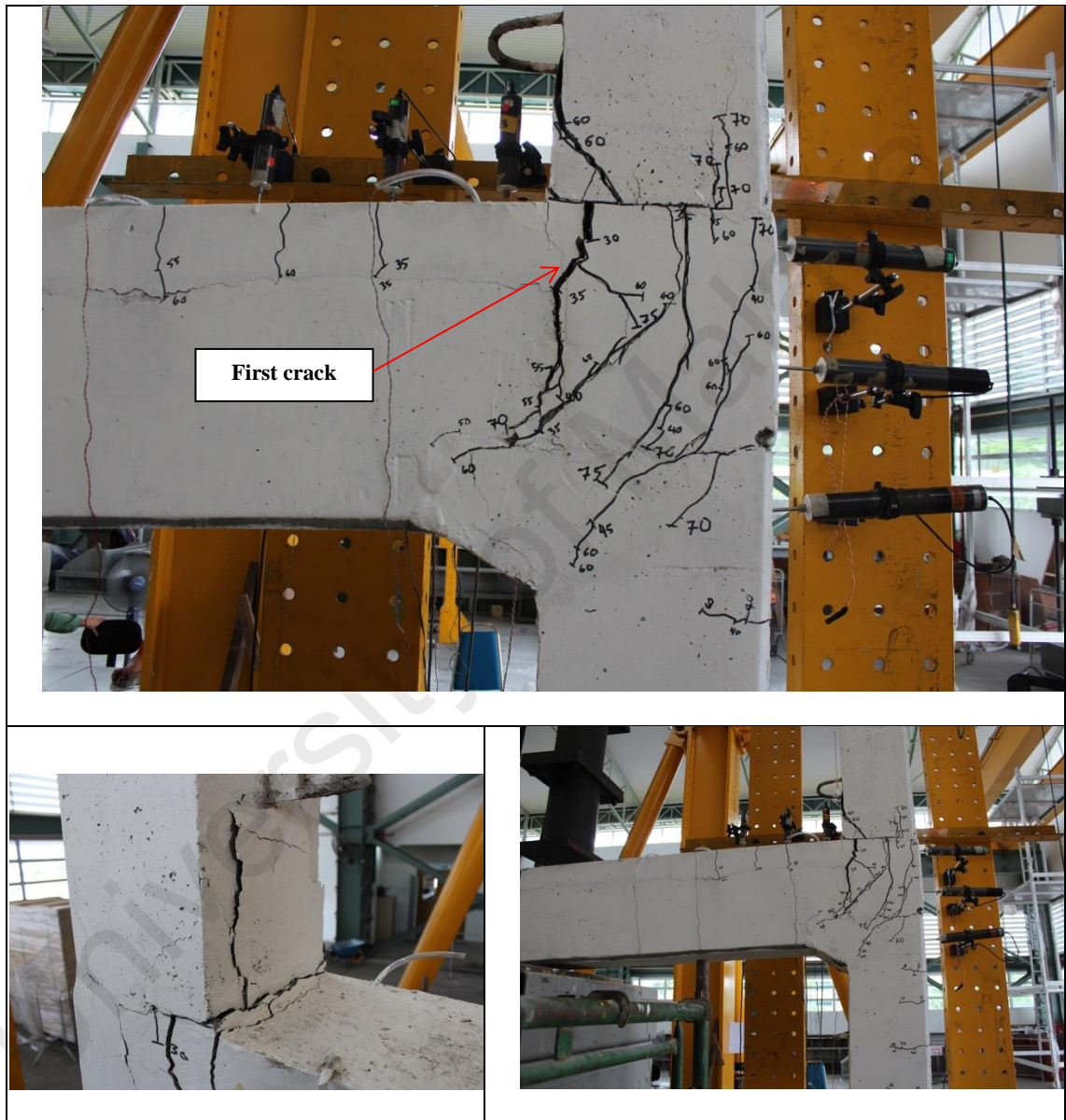
Figure 4.15: Failure mode for BHC2
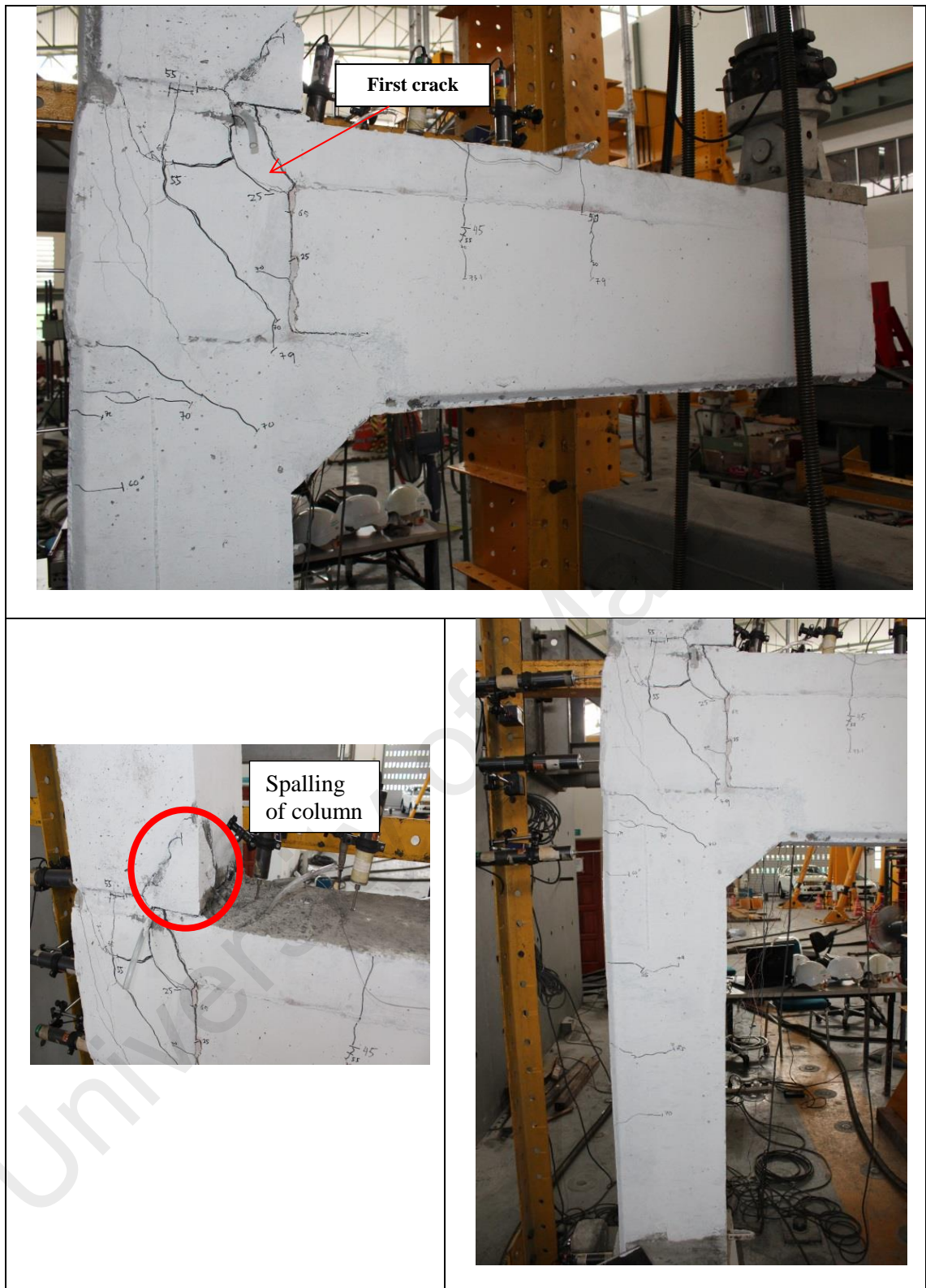
Figure 4.16: Failure mode for BHC3

Cracks formed in the panel, resulting in a slip of steel along column depth; this is a typical occurrence in which plastic hinges are developed in the panel or column (Wahjudi *et al.*, 2014). It can be noticed that cracks also occurred along plastic hinge length, l_p, in

connection zone. Shear cracks in the column, inclined horizontally at about 65°, are due to combined flexural tension and horizontal resolution of M over the beam's depth. For example, if M_{beam} = 80 kNm and h = 0.45 m, then H = 176 kN gives a shear stress in a column of 2.0 N/mm² whereas $M_{column} = M_{beam}/2$ = 40 kNm and bending stress = 8.2 N/mm², it is clearly sufficient to cause column damage.

4.9   Comparison between Theoretical and Experimental Values

The value of M_E and S_E are important parameters to be used in semi-rigid frame analysis (Note that; frame analysis is not in this thesis scope). The values for M_E and S_E for BHC2 and BHC 3 is higher compare to BHC1. This means the connection with detail T2 gives better result in terms of connection's overall performance.

By comparing the results obtained, it could be summarised that the experimental values are less than theoretical value as shown in Table (4.4) – (4.5). Differences of M_E values in BHC1, BHC2 and BHC3 are 40%, 28% and 24%, respectively. Meanwhile, differences for fixity factor, γ, are 43%, 30% and 24%, respectively. This can be explained that all tie bars in connections unable to achieve their full yield capacity due to effects of bond-slip and slippage failure of flexural reinforcements as seen in load strain curves in Figure 4.12. Besides that, a single sided test also contributes to the different results obtained compared to theoretical results, because connections limit the strength of the connection itself. Connections will achieve full capacity because tie steel in H-frame is fully effective; thus, will increase structure's stiffness.

Table 4.4: Theoretical and experimental test moments and rotations

Connection	Theoretical values		Test values	
	Relative rotation, ϕ_c (m.rad)	Required moment capacity, M_{ER} (kNm)	Beam-end rotation at E, ϕ_E (m.rad)	Beam-end moment at E, M_E (kNm)
BHC1 (T1)	4.75	89.65	10.0	54
BHC2 (T2)	5.58	97.33	9.1	70
BHC3 (T2)	5.58	98.45	8.3	75

Table 4.5: Theoretical and experimental test stiffness and fixity factor

Connection	Theoretical values		Test values		
	Rotational stiffness, S (kNm/m.rad)	Fixity factor, γ	Beam-end second stiffness at E, S_E (kNm/m.rad)	Ratio K_S, at E	Fixity factor, γ
BHC1 (T1)	18.87	0.69	5.40	0.47	0.39
BHC2 (T2)	17.44	0.67	7.69	0.67	0.47
BHC3 (T3)	17.65	0.67	9.04	0.78	0.51

4.10 Comparison between Proposed Connections and Elliott's Single Sided Connections

Figure 4.17 displays the relationship between moment ratio, M_E/M_R, and stiffness ratio, $K_{S,E}$, of Elliott's connections (Elliott *et al.*, 2003). Results for BHC1, BHC2 and BHC3 are drawn on the graph to provide a comparison between these specimens and Elliott's single sided connections. Based on the plotted graph, they are in good agreement, having greater moment ratio (M_E/M_R) values than Elliott's single sided connections, but less stiffness for the same M_E/M_R values. When the first crack appeared in the connection zone, it reduced axial stiffness and limited axial strength of the overall structure. Besides that, from experimental observation, initial cracks developed earlier at the beam-column interface. Furthermore, rotational stiffness does not exist as a real stiffness; it is a theoretically convenient value that relates strength of connector M_{RC} to S_E (Elliott *et al.*, 2003).
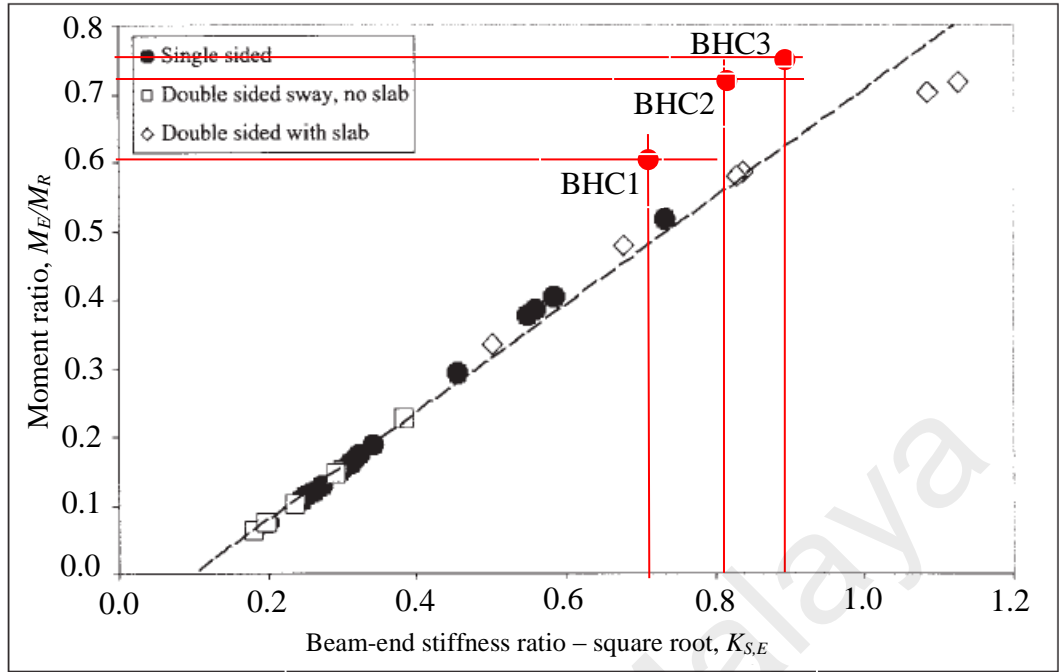
Figure 4.17: Comparison between BHC1, BHC2, BHC3 and Elliott's single sided connections.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATION

5.1 Introduction

In this research, connections were examined for structural performance as measured by forces and displacements from which moment-rotations were calculated in the beam-to-column connection. The works developed in this study were built based on current practices in Malaysian construction industry (i.e. standard Public Work Department (PWD), Malaysian design). The size of the members and the reinforcement of the precast column and beam and their strength were chosen to simulate an actual building frame as close as possible. This chapter concludes the findings of experimental tests carried out on three specimens of the proposed connection.

5.2 Conclusions

Generally, all the research objectives have been achieved through this research. Three full scale tests were successfully carried out on precast concrete beam-to-column connections, which had a partly hidden corbel with different anchorage negative moment top reinforcements (2 nos. T20 bars) into the column. The connections are formed using 1500 mm long x 450 x 300 mm deep single sided beams and a 3000 mm high x 300 mm x 300 mm edge column containing 4 nos. T25 main reinforcement bars.

The conclusions of this research and experimental testing could be summarized as follows:

1) Evaluation of the proposed precast beam-to-column connection's classification was performed using moment vs rotation (M-ϕ) data and plotted graphs. The moment, M_E and stiffness, S_E requirements of the connector at the end of the beam are determined using the intercept of the M-ϕ plot with the beam-line.

i. The calculated moment of resistance of the connections, M_{RC} = 111 kNm, 122 kNm and 124 kNm.

ii. The ultimate moment, M_U is in the range of 105 kNm to 124 kNm. Meanwhile, the ratio of connector failure load, M_U / M_{RC}, are range between 0.86 – 0.99.

iii. The beam-end moment, M_E from 54 - 75 kNm and stiffness S_E between 5.40 to 9.04 kNm/m.rad. Then $K_s = S_E$ /4EI/L, 0.47 – 0.78.

iv. Precast connection BHC2 exhibited considerably higher ductility (21%) compared to BHC1. The use of 90° bend bars for anchoring beam's longitudinal reinforcement is found to improve connection behaviour. Moreover, the ductility of precast connections can be further improved by increasing anchorage length and type of anchorage detailing.

2) Fixity factor, γ, (i.e. γ = 0 for pinned and γ = 1 fully rigid) for these connections were 0.39, 0.47 and 0.51. This place the connections in Zones II and III, from five classifications of semi-rigid joints based on Monforton's Fixity Factor.

3) The crack behaviour of these proposed precast connections (flexural cracks) was initiated at the column to precast beam joint interface due to relative strength and stiffness weakness of the two different materials in the joint. Flexural cracks comprise of three stages. First, cracks appear at the mode of failure for *in-situ* infill, then they grow and widen in a diagonal pattern and finally, they split at maximum load. In corbel, these cracks developed in a more gradual manner and involved the formation of a plastic hinge in the connection region. Precast connections show anchorage failure, categorised as side split and bond-slip failures.

4) Experimental values for beam-end moment, M_E and fixity factor, γ are less compared to theoretical values for BHC1, BHC2 and BHC3. The differences are 40%, 28% and 24% for M_E, and 43%, 30% and 24% for fixity factor, respectively. This difference can be explained by effects of bond-slip and slippage failures of

flexural reinforcements in the connections; single sided test contributes in reducing stiffness and limits the strength of connections.

The stiffness of the connection is greatly influenced by bond characteristics of the continuity of tie bars placed in the cast in situ topping. The overall experimental result given in this research shows that the proposed connection (BHC-T2) can be classified as semi-rigid. The value of M_E and S_E can be used in semi-rigid frame analysis. In addition, fixity factor value can be classified as semi-rigid under zone III (semi-rigid with medium strength).

5.3   Recommendation for Future Work

The future research of precast concrete beam-to-column connections could be focused into two parts which are experimental and analytical works.

1. Perform the laboratory work in double sided test (i.e. H frame test) to simulate actual practice in order to obtain the true response and accurate strength and stiffness of the connections.

2. Carry out the frame analyses using established equations (i.e Elliott *et. al.* 2003) to develop a guideline for using this type of connection.

3. Develop precast corbel connections by using a different type of anchorage detailing to improve the degree of connections rigidity.

4. Measure the crack width opening in the connection zone in order to be able to interpret the effect of jointing in connections.

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