FULL SCALE TESTING OF PRECAST BEAM TO COLUMN CONNECTION USING BILLET CONNECTOR AND BEAM HALF JOINT SUBJECTED TO REVERSIBLE LOADING

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ABSTRACT

Precast beam to column connection is an important element in precast concrete structure, which has significantly influenced the overall structural performance. This connection is used to transfer the shear, bending moment and sometimes torsion between the precast components. This research is to determine the moment resistance and moment rotation characteristic of new proposed precast beam to column connection through three (3) full-scale experimental studies. The specimens used for the testing are similar in geometrical and material properties. From the moment rotation characteristic, it is possible to extract the rotational stiffness, moment capacity and ductility of the connection. The experimental results were validated with existing analytical methods and the connection classification is determined. It is found that the ultimate moment of the connection, M_U is greater than the calculated moment resistance, M_{RC} for all specimens with the average value of M_U/M_{RC} is 1.21. All the specimens failed beyond the beam-line which means that the connection has sufficient ductility and achieved required strength to be considered as a semi-rigid connection and might be considered as a fully rigid. Based on the connection classification system according to Monforton's fixity factor, this connection falls in zone III, which is semirigid connection with medium strength. The analytical model overestimates the experimental results due to the omission of mechanical parts contribution such as the horizontal bolt, dowel and billet in calculating the rotation. For failure mechanism, all specimens exhibit plastic hinge formation in the beam at the column's face which means that the ultimate moment resistance of the beam was reached.

ABSTRAK

Sambungan rasuk-tiang merupakan elemen terpenting dalam struktur konkrit pra tuang dimana kelakuannya mempengaruhi keseluruhan prestasi struktur bangunan konkrit pra tuang. Sambungan ini berperanan untuk menghantar ricih, lenturan momen dan kadangkala kilasan diantara komponen-komponen pra tuang. Kajian tesis ini adalah untuk menentukan momen rintangan dan ciri-ciri momen putaran (*moment rotation*) untuk sambungan konkrit pratuang yang dicadangkan melalui kaedah eksperimen berskala penuh. Spesimen yang digunakan untuk ketiga-tiga ujikaji ini adalah sama dari segi geometrik serta ciri-ciri bahan. Daripada ciri-ciri momen-putaran, kekakuan putaran, kapasiti momen dan kemuluran sambungan dapat diestrak melalui eksperimen ini. Disamping itu, keputusan eksperimen turut disahkan dengan kaedah analitikal dan jenis sambungan dapat ditentukan. Hasil daripada ujikaji ini, didapati momen maksimum, M_U bagi sambungan pra-tuang ini lebih besar berbanding momen rintangan teori (M_{RC}) iaitu dengan nilai purata M_U/M_{RC} sebanyak 1.21. Semua spesimen semasa gagal adalah melepasi garisan *beam-line* yang bermaksud sambungan tersebut mempunyai kemuluran yang cukup untuk mecapai kekuatan yang dikehendaki untuk dipertimbangkan sebagai sambungan separa tegar atau sambungan tegar. Berdasarkan sistem klasifikasi Monforton's fixity factor, sambungan pratuang ini berada dalam Zone III iaitu sambungan separa tegar dengan kekuatan sederhana. Keputusan daripada eksperimen adalah dibawah anggaran kaedah analitikal disebabkan oleh sumbangan komponen mekanikal (*bolt* mendatar, *dowel* dan *billet*) tidak diambil kira dalam pengiraan putaran. Bagi mekanisma kegagalan, semua specimen menunjukkan kegagalan engsel plastik pada rasuk berhampiran muka tiang yang membawa maksud momen rintangan maksimum bagi rasuk telah pun dicapai.

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbols

A_S	:	Area of steel
d	:	Effective depth
f_{cu}	:	Compressive strength of concrete
f_y	:	Tensile strength of reinforcement
E	:	Young's Modulus
E_c	:	Young's Modulus of concrete
E_s	:	Young's Modulus of steel
I	:	Second moment of Area
kN	:	KiloNewton
K_S	:	Stiffness factor
L	:	Beam span
m	:	Beam line gradient
m	:	Meter
mm	:	Millimeter
M	:	Moment
M_E	:	Allowable moment capacity
M_{ED}	:	Allowable design moment capacity
M_{ER}	:	Required moment capacity
M_{RC}	:	Moment resistance of the connection
N/mm^2	:	Newton per millimeter square
P	:	Load
sw	:	Selfweight
S	:	Rotational stiffness

S_E	:	Secant stiffness
z	:	Lever arm
l_e	:	Embedment length of reinforcement across column
L_p	:	Plastic hinge length
γ	:	Monforton's Fixity Factor
δ	:	Deflection
ω	:	Uniformly distributed load
ϕ	:	Rotation
ϕ_C	:	End relative rotation
Δ_u	:	Displacement at ultimate load
Δ_y	:	Displacement at yield load
ε_{cr}	:	Strain at cracking load
ε_y	:	Strain at yield load
ε_u	:	Strain at ultimate load

Abbreviations

BIC	:	Billet Connection
BS	:	British Standard
CIDB	:	Construction Industrial Development Board
CREAM	:	Construction Research Institute of Malaysia
IBS	:	Industrialised Building System
JKR	:	Public Work Department of Malaysia/ Jabatan Kerja Raya
KLIA	:	Kuala Lumpur International Airport
LVDT	:	Linear Variable Displacement Transducer
MS	:	Malaysian Standard
PKNS	:	Perbadanan Kemajuan Negeri Selangor
RHS	:	Rectangular Hollow Section
SHS	:	Square Hollow Section
UB	:	Universal Beam
UC	:	Universal Column
ULS	:	Ultimate Limit State

LIST OF APPENDICES

CHAPTER 1: INTRODUCTION

1.1 Background

Beyond 2010, the government of Malaysia is moving towards adopting Industrialised Building System (IBS) in our modern construction industry. IBS is defined as a construction system in which components are manufactured in a factory, on or off site, positioned and assembled into structure with minimal additional site work (CIDB, 2003).

IBS has been introduced to our construction industry since 1966, but the usage of IBS is low and not so popular compared to cast in situ construction at that time. As a result, IBS has been ignored until the Government of Malaysia reintroduced it again due to its benefits. As the starting point, the Public Work Department of Malaysia (JKR) is enforced to use the IBS at least seventy percent in their building design and encouraged the engineers, architects and contractors from private sectors to use this new system.

The implementations of IBS are intended to reduce the unskilled workers, less wastage, less volume of building materials, increased environmental and construction site cleanliness and better quality control among others. Besides, it also promotes a safer and more organised construction site, and reduces the completion time of construction. As a result, the buildings like Petronas Twin Towers, Putrajaya, KL Sentral and Kuala Lumpur International Airport (KLIA) have chosen this system instead of conventional method.

To achieve the usage of IBS, the pre-cast concrete system is used. The pre-cast concrete system employs the use of prefabricated components which are manufactured using industrial process and assembled and erected into structures at sites. Pre-cast building components have received a wide attention in the building construction and have achieved a great deal of success in the modern day construction. Basically, there are three (3) types of pre-cast concrete structure which are the wall frame, the portal frame and skeletal frame. The skeletal frame mainly used for commercial offices, car parks, shopping centers, schools and so on. While for portal frames, they are limited for warehouses and wall frames are used for hotels, modular apartments etc.

The connection between pre-cast concrete components plays an important role in determining the success of pre-cast concrete structures. The connection provides connectivity among the precast element, it ensures the strength and rigidity of the structure and its resistance to applied load. For a precast skeletal frame structure, connection between the beam and column is very important, where the design and analysis of precast skeletal structures is greatly influenced by this connection (Elliott *et al.* 1998). This connection will govern the overall performance of the precast concrete frame.

1.2 Problem Statements

Precast concrete structures with pinned connections are widely used throughout the world. It provided simple in detailing and construction where the element to element bearing is the simplest form of pinned connection. However, the structural depth for precast connection is deep and it needs to be used together with bracing or shear wall for lateral stability. The development of moment connection can minimize the structural depth and reduce the use of bracing elements.

The most popular pinned connection used is corbel connection. Corbel is not preferable by the architects due to its limitation in appearance. Thus, the architectural demands have led to the design of invisible or hidden connection where the entire connection is contained within the beam. The design of connection without corbel is an approach to fulfill the architectural requirement. In addition, there is a need for a higher capacity precast beam to column connection to meet moment connection requirement.

Currently, the experimental data for moment connection detail for precast beam to column connection is still lacking. The data and the reliable behavior can only be accessed by laboratory testing and proven performance. Thus, more experimental works should be carried out to overcome these problems and also to obtain relevant data especially for precast beam to column connection. This study carried out full scale testing in order to develop a connection with similar behaviour of monolithic one.

1.3 Objectives of Study

This study was performed in order to achieve the following objectives:

i. To determine the moment resistance of proposed precast beam to column connection through laboratory testing.
ii. To determine the connection classification of proposed precast beam to column connection based on Connection Classification System according to Monforton's Fixity Factor.
iii. To validate the experimental results with analytical/ theoretical model result.
iv. To study the behaviour of precast beam to column connection in terms of moment-rotation (M-ϕ) relationship, load displacement relationship, failure modes and crack patterns.

1.4 Scope of Works

This study are focused on:

i. A new proposed precast beam to column connection using billet connector together with beam half joint. This connection was designed based on recommendation of BS8110:1997.

ii. Experimental works of proposed precast beam to column connection. A total three (3) specimens with similar geometrical and material properties were tested. The repetitive testing were carried out in order to confirm the results and the average value of the tested parameters.

iii. Behaviour of proposed precast beam to column connection is obtained from experimental works.

iv. Verification of analytical model of precast beam to column connection chosen from study by Ferreira (1993).

1.5 The Structure of Thesis

Overall, this thesis consists of five (5) main chapters. The chapters are Introduction, Literature review, Research methodology, Results and Discussions, Conclusions and Recommendations.

The briefing of the topics, the objectives, scope of work, problem statements are included in Chapter 1. In Chapter 2, the information regarding precast beam to column connection, the previous research in this topic and types of connection are explained. Chapter 3 presents the research methodology involved in order to achieve the objectives.

The discussion about the results, analysis of results and errors occurred are described in Chapter 4. Then, it followed with Chapter 5 which is the conclusions and recommendations. All the whole research carried out in this study and its results are concluded here. This chapter also consists of the recommendations and suggestions as the guide for the next future researcher who have an interest to do research in this scope of topics.

CHAPTER 2: LITERATURE REVIEW

2.1 Background

2.1.1 The History of IBS

The history of IBS began in early 1624 where panelised timber houses were shipped from England to new settlement in North America. Then, the Crystal Palace in Hyde Park, London was built in 1851 for Great Exhibition and Eiffel Tower in 1889 for Paris World Expo and French Revolution Centenary. Whilst in Malaysia, the IBS concept was introduced in 1966 where two pilot projects on IBS were launched by the Government of Malaysia. These two pilot projects are namely the Pekeliling Flats Kuala Lumpur and the Rifle Range Road Flat in Penang. Both projects applied the precast concrete elements to build these high rise low cost flats. Then, it followed by housing projects under Perbadanan Kemajuan Negeri Selangor (PKNS), a state government development agency in 1981 till 1993. PKNS acquired precast technology from Praton Haus International based in Germany (CIDB, 2003).

To date, the usage of IBS as a method of construction is evolving after four (4) decades (1960-2000) in lukewarm situation. Many private companies team up with foreign experts to offer IBS solutions. Local IBS players were also mushrooming. Many private projects started to use IBS which previously dominant by government projects. Current construction industry looking for better method of construction that offers quality, safety, time and cost reduction, and also aesthetic value to the building constructed. In extension of this, Malaysian construction industry is now moving towards modernization, mechanization and industrialization of precast concrete technology.

2.1.2 Types of IBS

Basically, there are many types of IBS and these can be categorized based on its construction's types. In Malaysia, Construction Industry Development Board (CIDB) has classified IBS into five (5) major categories which are:

i. Precast concrete framing, panel and box systems
ii. Steel formwork systems
iii. Steel framing systems
iv. Prefabricated timber framing systems
v. Blockwork systems

The application of these types of IBS in construction industries are shown in Table 2.1 below.

Table 2.1: Types of IBS and its applications

Types of IBS	Application
Precast concrete framing, panel and box systems	Precast columns, beams, slabs, walls, 3D components (staircases, toilets, balconies, lift chambers, refuse chambers), and lightweight precast concrete as well as permanent concrete formworks.
Steel formwork systems	Tunnel formworks, beams and columns moulding forms, tilt up systems, slab moulding forms and permanent steel formworks (metal decks).
Steel framing systems	Steel beam, columns, portal frames, roof trusses
Prefabricated timber framing systems	Timber frame, timber roof trusses
Blockwork systems	Interlocking concrete masonry unit (CMU), lightweight concrete blocks

2.2 Precast Concrete System

To achieve the usage of IBS, the precast concrete system is used. The precast concrete system employs the use of prefabricated components which are manufactured using industrial process and assembled and erected into structures at sites. Precast building components have received a wide attention in the building construction and have achieved a great deal of success in the modern day construction.

Basically, there are three (3) types of precast concrete structure which are wall frame, portal frame and skeletal frame. The skeletal frames are mainly used for commercial offices, car parks, shopping center, schools and so on, While for portal and wall frames, they are limited for warehouses, industrial buildings, hotels, modular apartments etc.

The application of precast concrete systems has introduced many advantages in construction industries. The advantages are the reduction of the construction period, good quality, low sensitivity to weather conditions, reduction of manpower on site and the possibilities to achieve greater span through the use of pre- tensioning method (FIP Commission on Prefabrication, 1986). However, to remain competitive, precast must be simple and fast in erection. Thus, the development of an efficient connection is very important.

2.3 Precast Concrete Connection

Precast concrete construction requires the presence of connection for assembling phase and to give the construction monolithic quality required for strength and durability. The connection design and realization have always presented the main difficulties in precast concrete construction (Song, 2004).

According to Trikha, *et al.* (2004), connection can be defined as the component that provides connectivity amongst more than two precast elements assuring rigidity of the structure and its resistance to the applied loads. The connection between precast concrete components plays an important role in determining the successful of precast structures where its behaviour affects the constructability, stability, strength, flexibility and residual force in structure. In addition, the connection plays key role in the dissipation of energy and redistribution of loads when the structure is loaded (Dolan, *et al*. 1987).

Connection is used to transfer load, provide strength and stability to the structure. The main structural connections that consist in precast concrete structure especially in skeletal frame are beam to slab connection, beam to column connection, wall to frame connection and column splices including the foundations. Among these connections, beam to column connection is the most important connection in precast skeletal frames. They are thought of by the profession at large as being difficult to specify, design and construct, especially those which are hidden within the beam (Elliott, 2002).

2.3.1 Criteria for Connection

As stated earlier, precast concrete connection is an important element in precast concrete structure, where its behaviour governs the performance of precast concrete structure. The precast connections must fulfill certain requirements or criteria to make it successful. Waddell (1974) has listed down some important properties that help connection become successful. The criteria are as follows:

i. It must be structurally adequate to perform at both service load and ultimate load, taking into account all possible loading conditions of the reactions, and restrained rotation that may cause moment in the connections. Good engineering decrees that the members fail before the connections, normally achieved by providing a safety factor in the connections ten percent (10%) higher than in the adjacent members.
ii. It must be compatible with the architecture of the structure, preferably not visible in the finished structure. If it must be exposed to view, it should be

neat and unobtrusive, non-rusting, and non-staining, and watertight. Edges and corners should be chambered and beveled.

iii. It must accommodate both manufacturing tolerances and erection tolerances. Both of these tolerances must be considered when determining the sizes of holes, sleeves, dowels, corbel and bearings, as well as erection clearances.

iv. It should be designed so that temporary bracing or connections can be made to hold the precast unit in place so the crane can be released as soon as possible. Tying up the expensive crane and crew for the extended time while the connection is welded, bolted otherwise completed is a needless expense.

v. It should be the most economical connection possible that fulfils the requirements of i, ii, iii and iv by considering all factors of precasting, handling, and erecting. This implies the use of standard manufactured items readily available in the market rather than specially made.

Besides, Elliott (1996) also listed down the criteria to satisfactory joint design. The criterias are:

i. Components able to resist ultimate design loads in a ductile manner

ii. Components may be manufactured economically and be erected safely and rapidly

iii. Tolerances for manufacturing and site erection do not adversely affect intended structural behaviour, or are catered for in a ‘worst case’ situation.

iv. Final appearance of joint must satisfy the visual, fire and environmental requirements

In addition, Vambersky (1990) has summarised the main criteria for the serviceability performance of join in terms of:

i. strength

ii. influence of volume changes

iii. ductility

iv. durability, including corrosion and fire protection

v. simplicity in fabrication and erection

vi. temporary loading conditions

vii. economy and appearance

Figure 2.1 and Figure 2.2 show the connections that have been proven unsuccessful. For Figure 2.1, the connection is unsuccessful due to no temporary bracing and difficult to construct at site while connection in Figure 2.2 fails due to high cost and difficult to position on site.

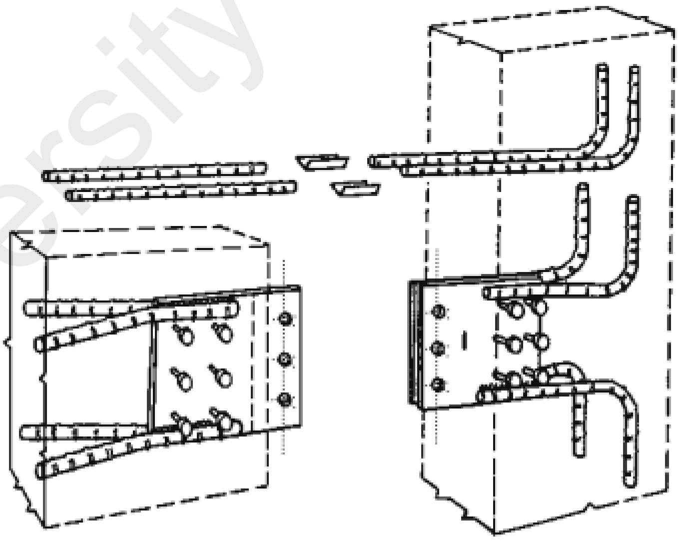


Figure 2.1: Unsuccessful type of precast beam to column connection (Elliott,1996)

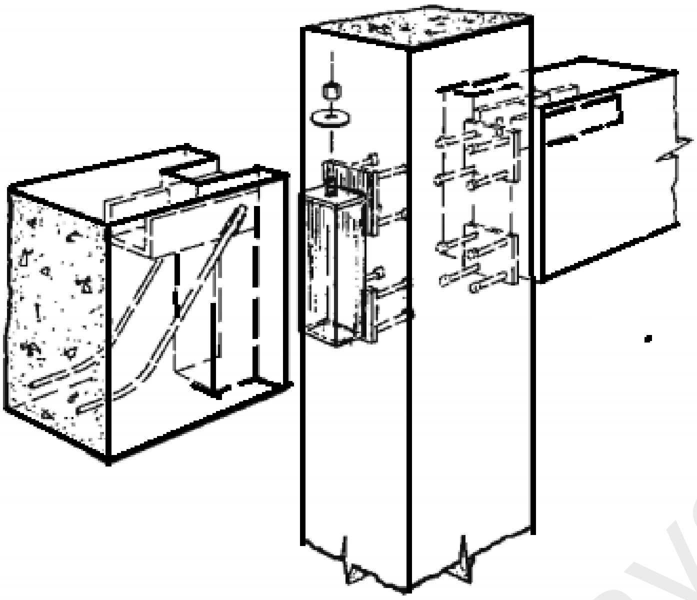


Figure 2.2: Unsuccessful type of precast beam to column connection (Elliott,1996)

2.4 Precast Beam to Column Connection

2.4.1 Types of Connection

In precast concrete structural framed system, the precast beam to column connections can be categorized into three (3) categories which are simple (pinned), semi-rigid and rigid (fixed) connections. These three (3) categories indicate the degree of moment to be transferred among the members. These behaviours are interpreted in typical moment rotation curve shown in Figure 2.3.

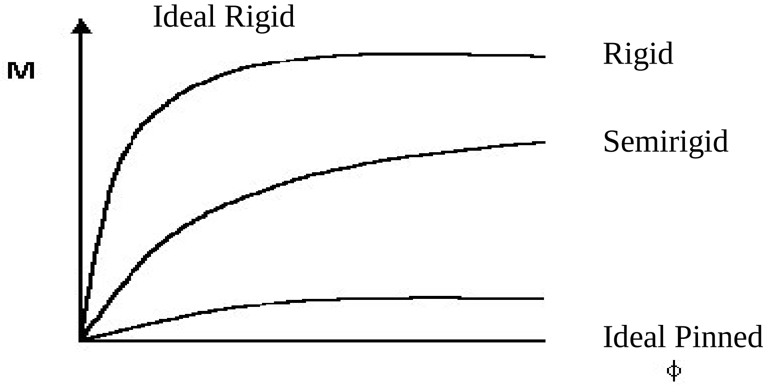


Figure 2.3: Moment rotation curve for connections

The rigid connection transferred full moment between members while simple connection transferred zero moment. The degree of moment transfer for semi rigid connection falls between rigid and simple connections. These connections are neither ideally pinned nor ideally fixed. The differences effect of connection types in terms of moment distribution in a structure is shown in Figure 2.4.

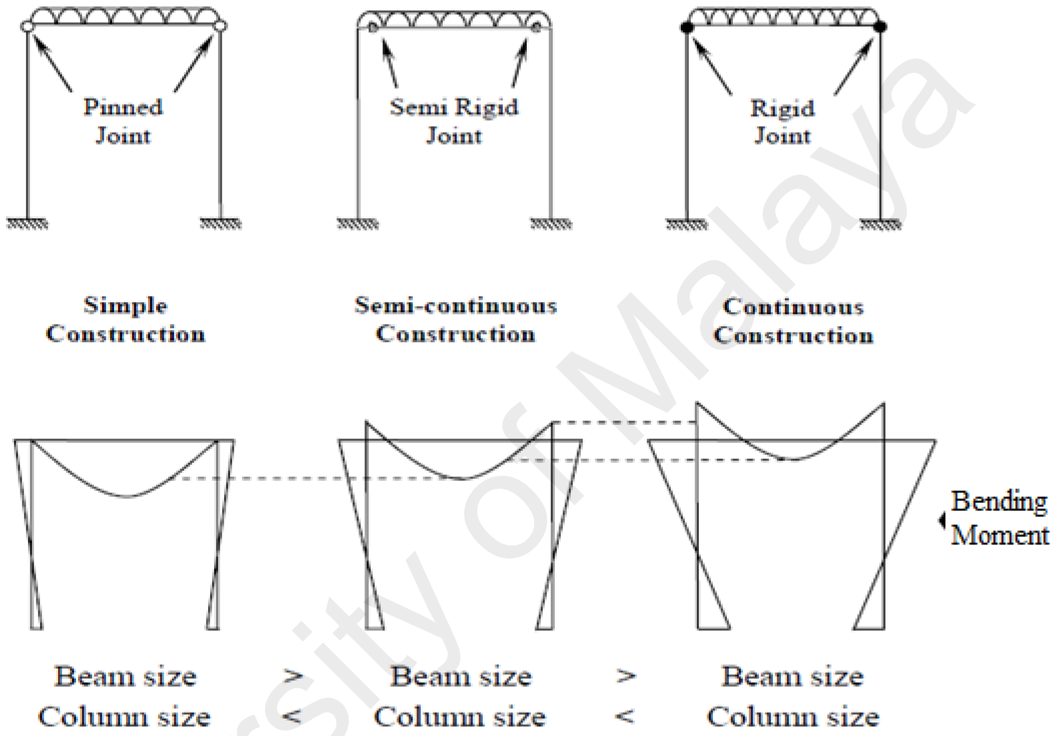


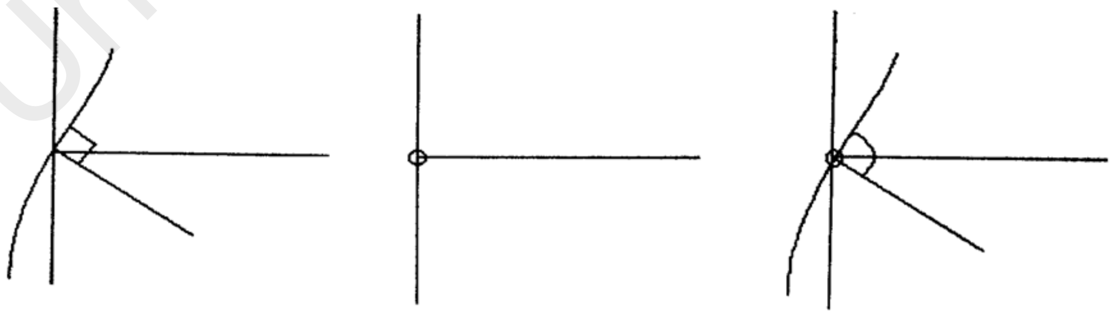
Figure 2.4: Effects of different connection types in terms of moment distribution (Kooi, 2005)

The characteristic of this connection are described in Table 2.2 below:

Table 2.2: Characteristic of connections

Types of connection	Characteristic
Simple connection	i. Simple connections (Figure 2.5b) are assumed to transfer vertical shear only. ii. Both rotational stiffness and moment resistance are small and may be reasonably neglected (can be assumed to approach zero value), leading to the concept of a pinned or hinged connection. iii. Such connection can be used only in non-sway frame where the lateral loads are resisted by some alternative arrangements such as bracing or shear wall.

	iv. Typically used in braced frames where strength rather stiffness govern the design. v. This connection lends themselves to simple detailing and construction, and maybe formed in the simplest manner by element to element bearing (Elliott,1996).
Semi-rigid connection	i. Semi-rigid (Figure 2.5c) connections are those fall between simple and semi rigid connection. ii. Such connections allow for a range of moment distribution in frames. It is neither zero (or very small) as in pinned connection nor fully moment transferred as in rigid connection. iii. It also does experience some degree of joint deformation and this can be utilized to reduce the joint design moments. iv. This connection has the true behaviour of the joint where certain flexural deformation is allowed for nominal rigid connection and certain degree of rotation is provided by nominal pinned connections. v. This type of connection may be used for both braced and unbraced frames, but in the latter case the influence of the connection flexibility on frame behaviour need to be considered. vi. They are also used in conjunction with other lateral load resisting systems in order to increase the safety and the performance of the overall structure.
Rigid Connection	i. Rigid connections (Figure 2.5a) are assumed to transfer full moment to the column without undergo any rotation between the members. Therefore, the moment rotation is always assumed to be zero. ii. Rigid connections are suitable for both braced and unbraced frames. It provides stiffness requirement especially in high rise and slender structure. iii. This connection also contributes in resisting lateral loads



a. Rigid connection b. Pinned connection c. Semi-rigid connection

Figure 2.5: Illustration of different range of connection’s behavior

2.4.2 Types of Precast Beam to Column Connection Used in Industry

2.4.2.1 Precast Concrete Connection with Embedded Steel Members

Connections with embedded structural steel members serving as haunches or bracket have been used for many years in precast concrete construction. The embedded steel or steel insert is used to transfer shear and axial force, and sometimes bending and torsion moment to the column (Elliott, 1996). Figure 2.6 shows the application of embedded structural steel in precast beam to column connection.

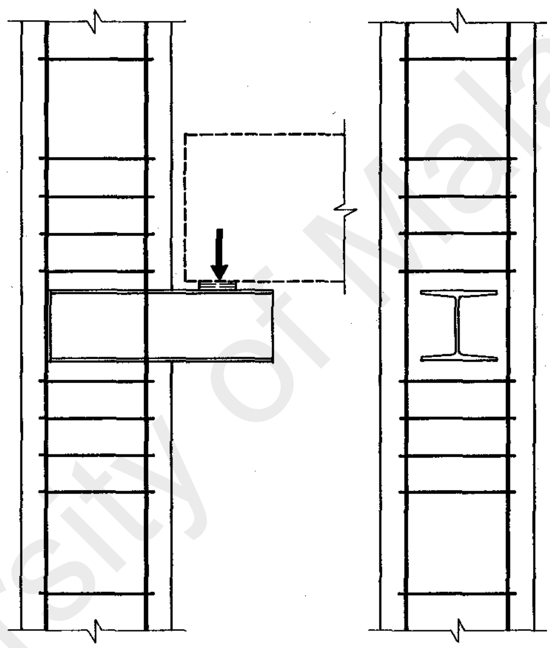


Figure 2.6: The application of embedded structural steel in precast beam to column connection (Marcakis and Mitchell, 1980)

Marcakis and Mitchell, (1980), have list down the advantages of this connection. The advantages are:

i. The strength of this connection is not greatly depend on the strength of the weld.
ii. Such connections do not usually require complicated reinforcement details.
iii. This connection can be easily designd to exhibit large ductility.

Normally, this embedded steel member will be used together with halving joint (half beam joint) in precast beam to column connection. Figure 2.7 illustrates the beam to column connection using halving joint and embedded steel member.

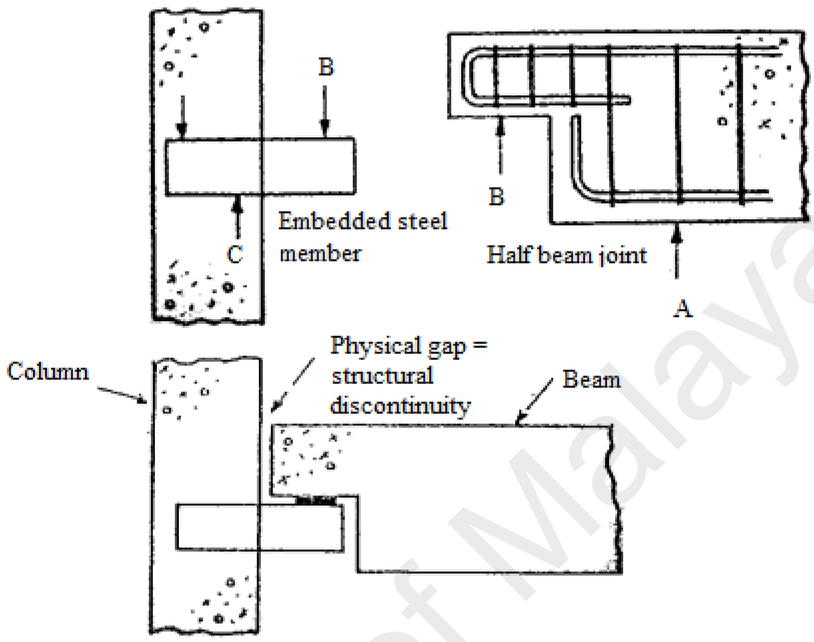


Figure 2.7: Beam to column connection using halving joints and cast in steel insert (Elliott, 1996)

Generally, the embedded steel can be adopted from various sections such as:

i. Universal column or beam (UC or UB)

ii. Rolled channel, angle or bent plate

iii. Rolled rectangular or square hollow section (RHS, SHS, etc)

iv. Threaded dowel or bolts in steel or and plastic tubes

v. Bolt in cast-in steel sockets

Again, Marcakis and Mitchell, (1980) in their research have stated that different sectional types of embedded structural steel member would affect the distribution of load and stresses, stiffness of the connection and failure modes. For example, a comparison results have been made between wide flange (UB or UC) and hollow section (RHS, SHS, etc). The connection with wide flange is stiffer than connection with hollow section. In terms of failure modes, vertical cracks are formed from both top and bottom flange of the wide flange section. This indicates that both flanges are effective in distributing the load. Compared to wide flange section, hollow steel section has only one loading surface to distribute the load. Therefore, wide flange section is more favorable in distributing the stresses in the connection.

Furthermore, if the hollow steel section had thin wall (not filled with concrete), the bearing of the concrete against the top wall of the steel member could cause several local bending. This affects the stresses concentrations in the concrete above the webs of the hollow steel section and reduce the effective width of the connection. This leds to a premature failure. Therefore, if the wall of a hollow steel section is not stiff enough, it should be filled with concrete to ensure a more uniform bearing stress which will enable the effective width to attain its maximum value.

In coherent with that, sometimes additional reinforcement is welded to the steel billet whereas this reinforcement is assumed can act both in tension or compression (refer Figure 2.8). The presence of welded reinforcement can increase the capacity and the stiffness of the connection.This is proven by experiment and it also done by Marcakis and Mitchell, (1980).

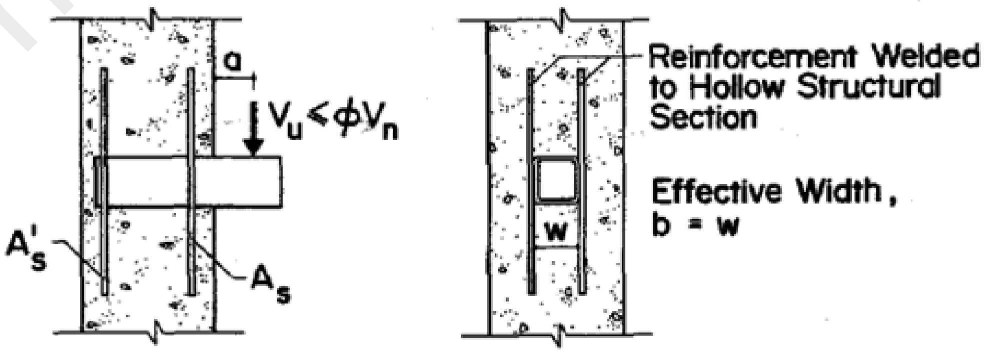


Figure 2.8 : Precast beam to column using solid billet with welded plate in beam

(Marcakis and Mitchell, 1980)

Besides that, there are several types of precast beam to column connection using structural embedded steel members. Such connections are:

i. Precast beam to column connection using solid billet with welded plate in beam (see Figure 2.9). This type of connection is a modified of Cazaly Hanger (PCI, 1988) where the cantilever beam is replaced by a deep narrow plate and the steel strap by two number of hooked end reinforcing bars welded to either side of the plate (see Figure 2.10).

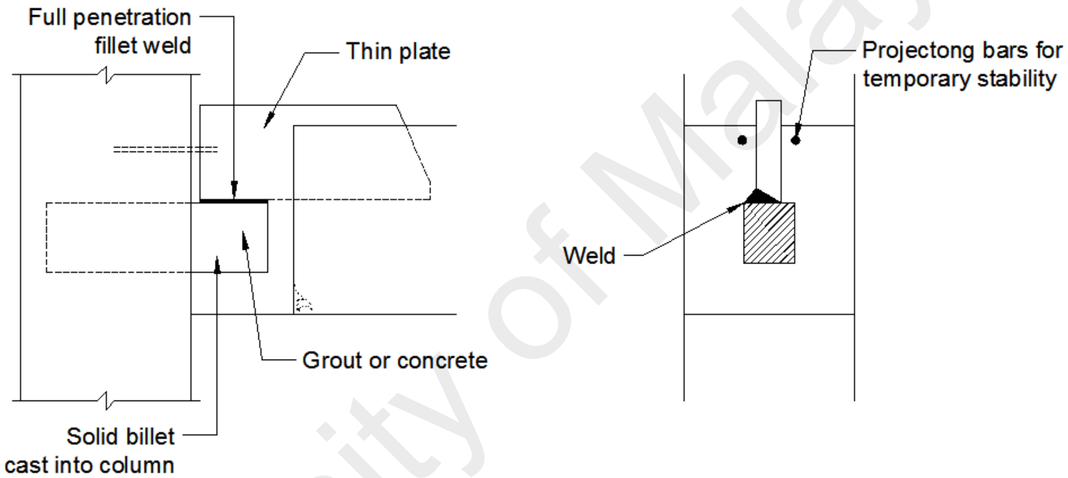


Figure 2.9: Precast beam to column connection using solid billet with welded plate in beam (Elliott, 1996)

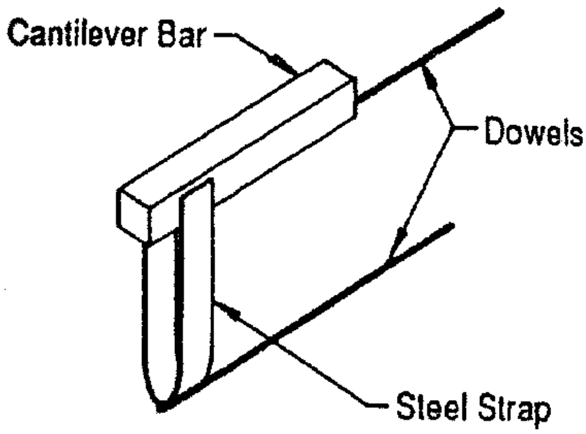


Figure 2.10: Basic components of Cazaly Hanger (PCI,1988)

The mechanisms of this connection are:

a. The projecting bars are arranged within the column width for temporary means. But, if these projecting bars are fully anchored to the column or continuous through the column, it is assumed that the projecting bars are fully stressed at limit state.
b. The beam is fully anchored such that the billet is also fully effective.
c. The contribution of the solid steel billet is then ignored due to limited strength of concrete infill at the bottom of the beam.

This simple connection can be designed to carry shear up to 500 kN. The connection requires site welding but the fixing is rapid.

ii. Precast beam to column connection using solid or hollow billet with top steel reinforcing bars (see Figure 2.11).

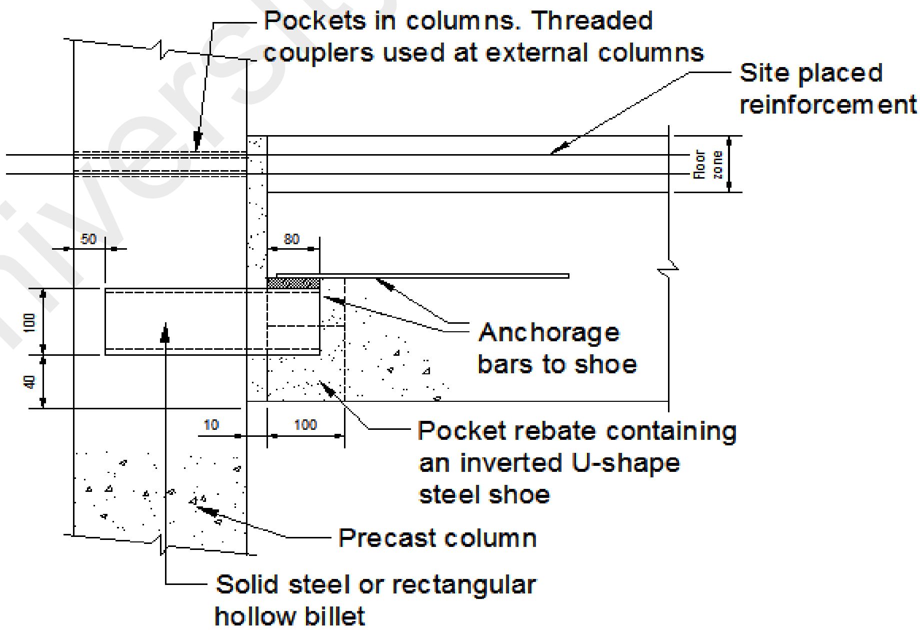


Figure 2.11: Precast beam to column connection using solid or hollow billet with top steel reinforcing bars (Elliott, 1996)

According to Elliott, *et al.* (1998), the billet connector is based on conventional steel haunch but without reinforcing bars welded to the sides of the box section. The connectivity among precast beam and column insert comes from direct frictional bearing with no positive mechanical action introduce between these both precast components. This connection is attempted to generate sagging moment where it is resisted by the addition of tie steel, bolted and/or welded plates. In addition, these addition of tie steel, bolted and/or welded plates also provide torsional stability to the connection.

iii. Precast beam to column connection using hollow section with threaded dowel and top angle steel

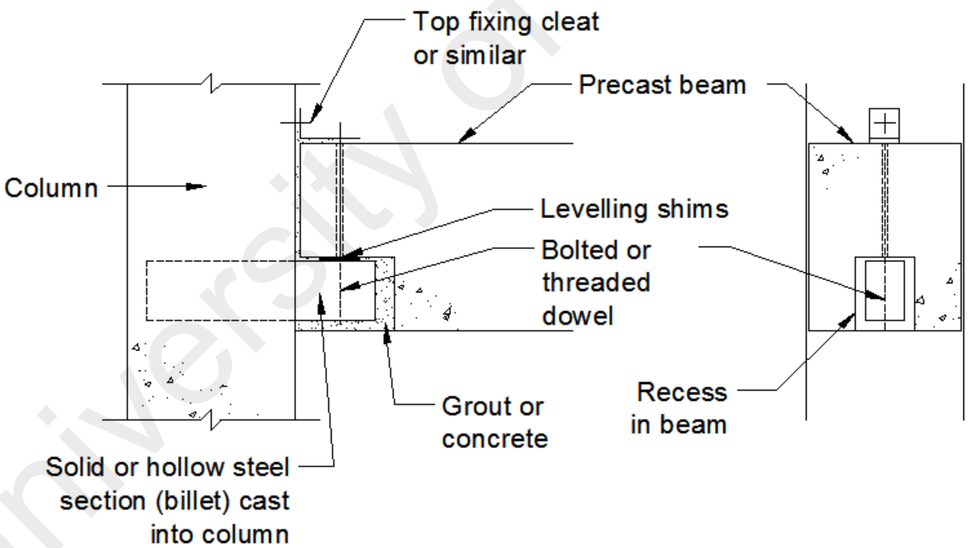


Figure 2.12: Precast beam to column connection using solid or hollow billet section with threaded dowel and top angle fixing (Elliott, 1996).

A threaded dowel is site fixed through a hole in the beam, supporting steel billet and secured to a steel angle (or similar) at the top of the beam. This illustration is shown in Figure 2.12. By doing this (top angle fixing), it would give immediate temporary stability effect to the connection and a positive mechanical tie between the precast components.

iv. Precast beam to column connection using open box and notched plate in beam

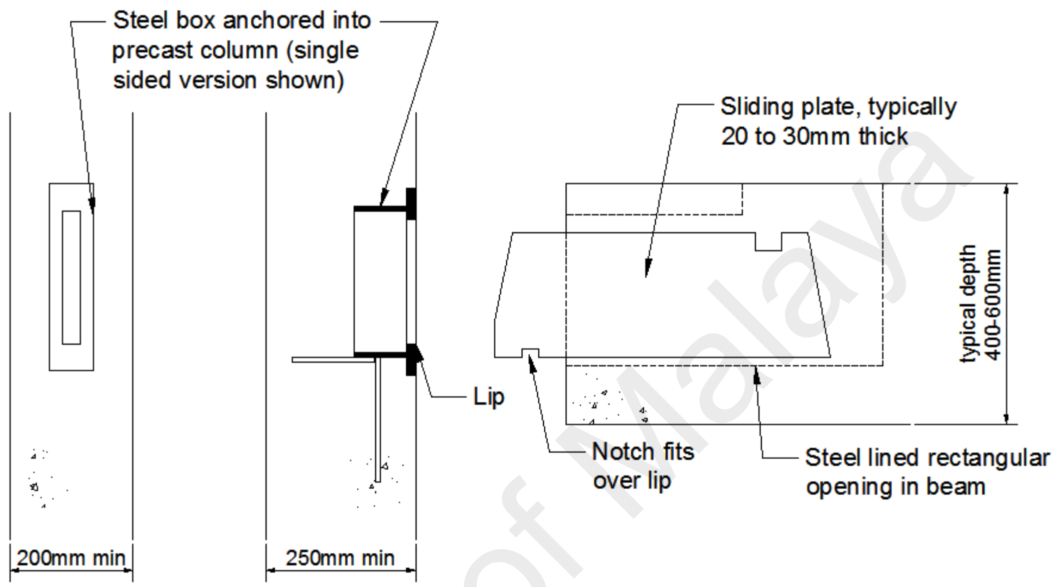


Figure 2.13: Precast beam to column connection using open box and notched plate in beam (Elliott, 1996)

This type of connection is a hidden beam end connection for gravity loads that eliminates the need for projecting column corbels. It provides a simple, efficient connection that allows designer a new freedom in creating clean, elegant lines in the completed precast concrete structure, fast in erection and it can function within normal building. The application of this type of connection are very wide, it can be used in all types of buildings where beam frames into the column. Such types of building are office buildings, schools, hotels, car parks and any other similar structures.

However, even though this type of connection provides many benefits to the building structure, but there is still a barrier to adapt to this system. The biggest barrier is the designer itself where not all the designers are widely familiar with this system.

From the illustration in Figure 2.13, a steel box cast into the precast concrete beam end while a sliding “knife” plate with a safety notch is cantilevered into a steel box is also been cast into the concrete column.

v. Precast beam to column connection using rolled H-section and bolted on cleat

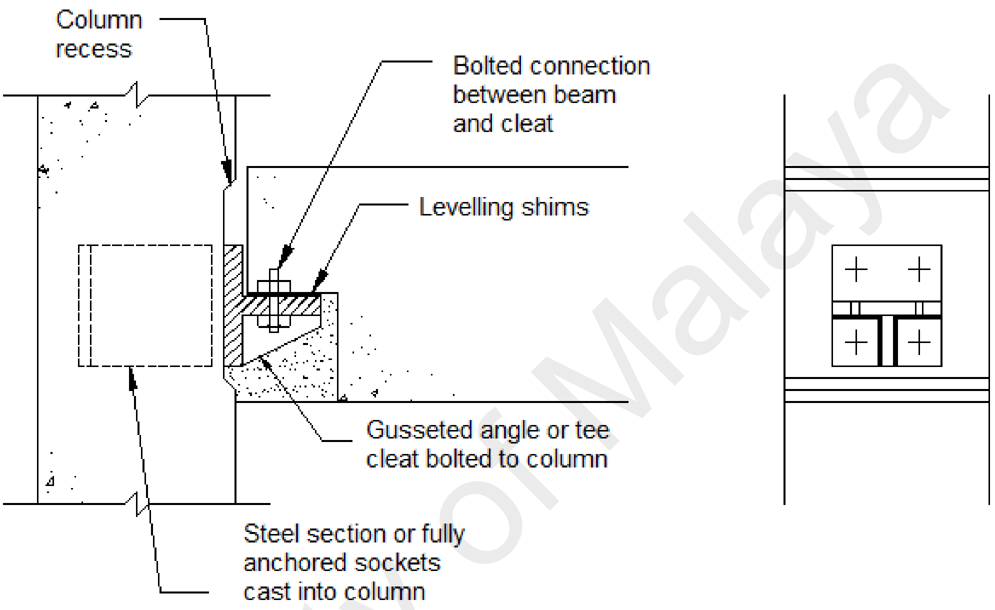


Figure 2.14: Precast beam to column connection using rolled H-section and bolted on cleat (Elliott, 1996)

For this connection, the connectors (cleat) introduce a third part linking beam and column units to avoid having mould penetration. Figure 2.14 shows how the column and the cleat connector may be cast in a mould (Elliott, 2002). Basically, the strength of this connection is greatly depends on a separate intermediate cleat. According to Elliott (1996), typically this cleat are rolled angle or fabricated rod gusseted for strength. To perform the connection, the cleat will receive a bolted connection to both beam and column components. Top fixing maybe excluded due to the stability provided by the bolt (at least two) group. This connection is expensive but safe to use. See also Figure 2.15.

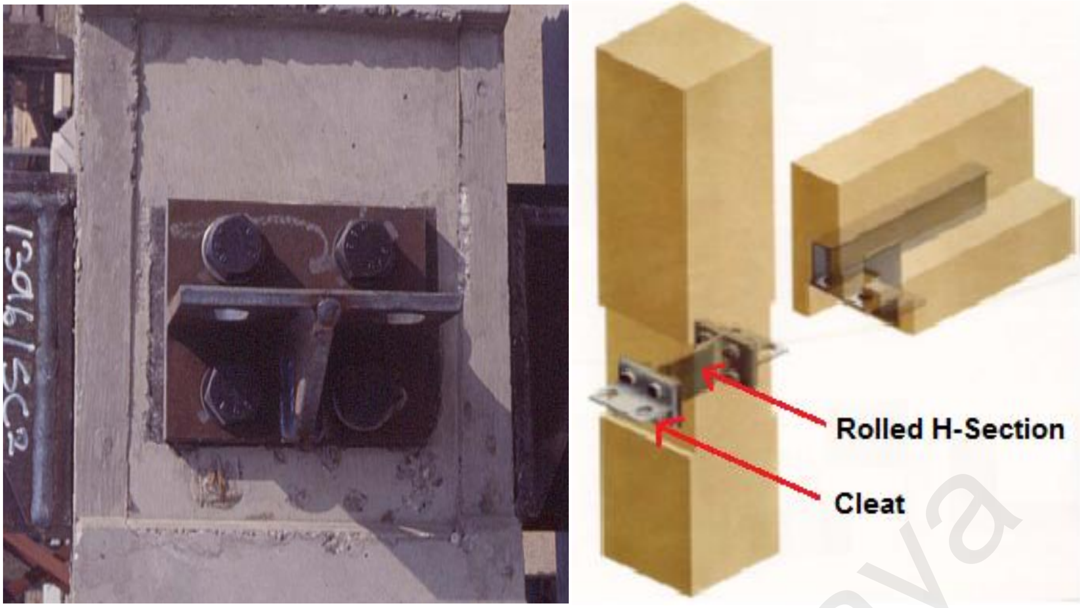


Figure 2.15: Precast concrete beam to column connection using rolled H-section and bolted on cleat.

2.4.2.2 Precast Concrete Connection using Corbel

Corbel is defined as short cantilever projection from the face of a column or a wall which support a load bearing components on its upper horizontal ledge. Column corbel is widely used in the Continental Europe and North America but not in United Kingdom. This type of connection is not preferable by the architects due to its appearance. Thus, the architectural demands have led to the design of invisible or hidden connection whereas the entire connection is contained within the beam.

Basically, connections by using corbel are pinned joint and it only transfer shear force to the column. If the corbel connection is applied to the frame structure, the frame needs to use the bracing, core or shear wall in order to maintain the frame stability. This is because the stability of the frame structure cannot be provided by the connection itself due to it negligible stiffness. These lead to uneconomical design of column and foundations.

However, it is possible to make this connection become semi rigid or rigid connection. Such ways are to have steel protruding from the precast elements, welding or overlapping the steel bars and achieving a moment resisting connection through in situ concreting the joints and leave threaded sockets in precast elements to receive nuts and bolts at site. Besides, it also be made by embedding steel sections or plates in the precast elements using steel angles and plates

According to Elliott, *et al.* (1998), since 1990, the tests on corbel are not widely carried out, the most testing are on welded plate and billet connector where some 24 tests have been carried out using those items within the period of time. Since the testing on corbel is not widely carried out, Ab Rahman, *et al.* (2006) had carried out some series of experiments by using corbel connections. Modifications and improvements have been made to the original corbel to make it semi rigid or rigid connection. As a result, these connections proved that the performance of connection in terms of stiffness, strength and moment resistance is slightly higher than conventional connection which is cast in situ and this connection also can cater moment compared to the existing corbel.

2.5 The Behaviour of the Connection

2.5.1 Moment Rotation (M-ϕ) Relationship

The moment rotation (M-ϕ) curve can interpret the behaviour of a connection whether it is rigid, semi rigid or pinned connection. This classification is due to the degree of moment to be transferred among the members. For example, the rigid connection transferred full moment between members while simple connection transferred zero moment. The level of transferred moment for semi rigid connection falls between rigid and simple connection. This is shown in Figure 2.16.

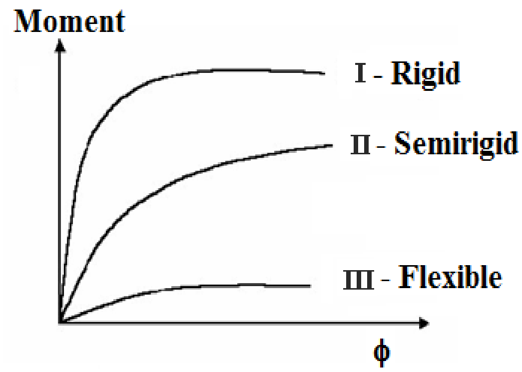


Figure 2.16: Moment rotation curve

The moment rotation curve also represented the stiffness of the connection (see Figure 2.17). The rotational stiffness for rigid connection is high while the pinned connection has small stiffness.

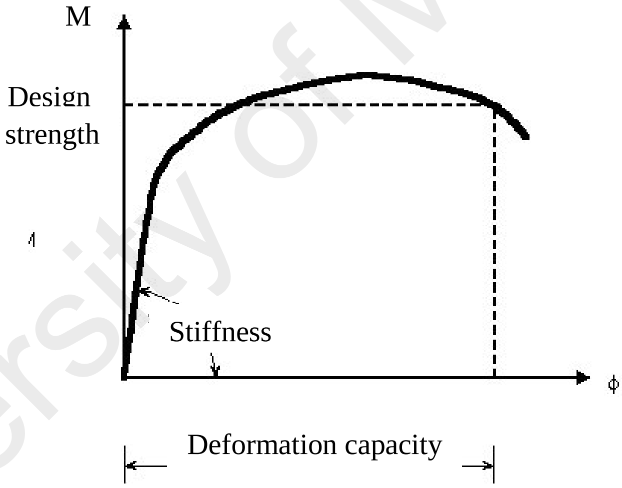


Figure 2.17: The interpretation of stiffness in moment rotation curve

For beam to column connection testing, moment can be obtained by multiplying the corresponding applied load with the distance of point load from the surface of the column (Elliott, *et al*. 2003). The applied load is an incremental load. The rotation of the connection can be obtained by dividing the corresponding vertical displacement with the distance of the Linear Variable Displacement Transducer (LVDT) or dial gauge from the surface of the column. The vertical displacement is usually measured using LVDT as the rotation is assumed to be very small (Leong, 2006). The moment and rotation for every incremental load is then plotted into a graph to produce the moment rotation curve. According to Ling (2004), the same procedures are also used to obtain the moment rotation curve for all numerical models with different values of distance of point load and dial gauge. Figure 2.18 shows the typical moment-rotation curve.

Ductility of beam to column connection is crucial in precast concrete construction. Therefore, crushing failure of concrete or brittle behaviour in connection must be avoided. This is shown in Figure2.19. The ductility of connection can be determined based upon factor of ϕu/ϕy, as shown in Figure 2.20.

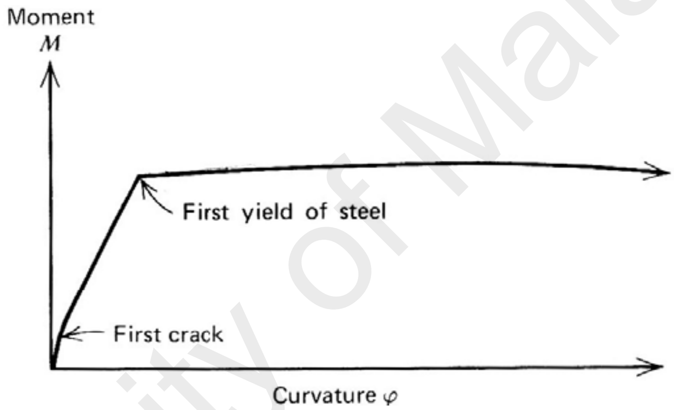


Figure 2.18: Typical moment-rotation curve (Park and Paulay, 1975)

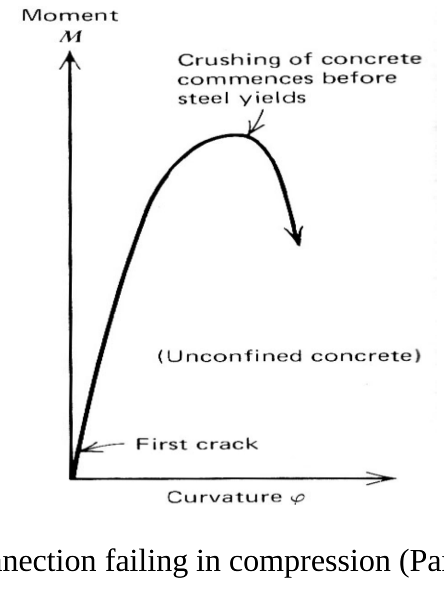


Figure 2.19: Connection failing in compression (Park and Paulay, 1975)

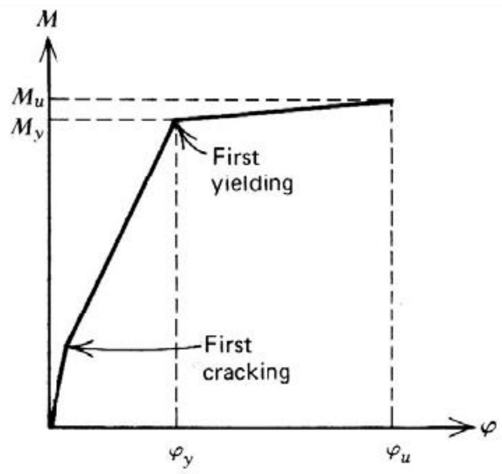


Figure 2.20: Moment-rotation curve (Park and Paulay, 1975)

2.5.2 Load Displacement Relationship

In precast concrete connection, the load displacement relationship is important to determine the characteristic of the connection. The load displacement curve can interpret whether the connections are ductile or brittle. A ductile connection is very important especially if the structure subject to seismic loading or the structure being loaded to failure in extreme event. This is because it is capable of undergoing large deflection at near maximum load carrying capacity to give warning of failure and prevent total collapse. Park and Paulay (1975) stated that this is due to the present seismic design philosophy relies on energy absorption and dissipation by post-elastic deformation for major survival in earthquakes. It is important to ensure that brittle failure will not occur. The graph of load displacement is shown in Figure 2.21

Ductile behaviour can be expressed by the ratio of the ultimate deflection (ductility factor), Δu, to the deflection at initial yield, Δy, or summarized as $\Delta u/\Delta y$ (Loo & Yao, 1995). According to Park (1988), the ductility factor may vary from 1 (full elastic) to 7 (ductile). Typically, the value for ductility factor is in the range 3 to 6. Department of Public Works (2002), had categorized ductility factor as shown in Table 2.3.

Table 2.3: Ductility factor for building structure

Performance level of building structure	Ductility factor
Full elastic	1 1.5 2 2.5
Partial ductile	3 3.5 4 4.5
Full ductile	5.3

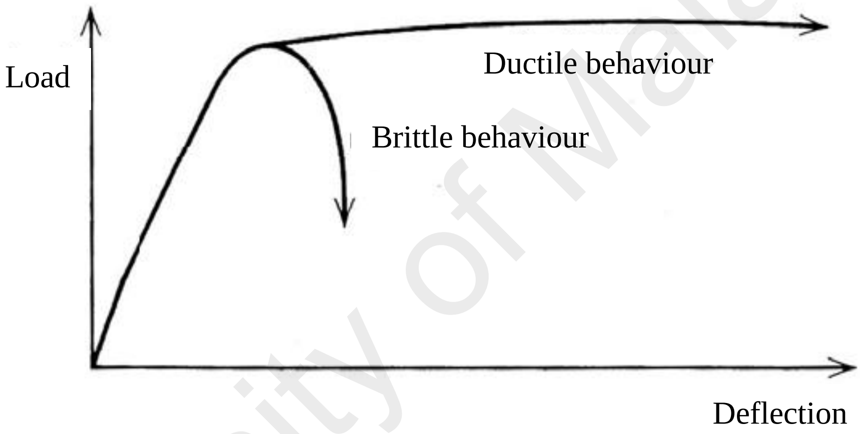


Figure 2.21: Load displacement curve (Park and Paulay,1975)

2.5.3 Beam Line Method

According to Elliott, *et al.* (2003), in order to determine the rigidity of the connection, a beam-line method can be used. This beam-line method represents the characteristic of M−ϕ behaviour of an elastic beam under a certain conditions of loading in a flexural cracked state (Figure 2.22). In order to determine the beam-line for a particular single beam subjected to uniformly distributed load (w) on a beam span (L), moment rotation diagram is constructed by considering the extreme condition. The conditions are:

i. First condition (to determine point A)

Pinned beam is assumed at point A which represent the rotation of the beam at the support under distributed load (when M=0, ϕ= $wL^3/24EI$).

ii. Second condition (to determine point B)

Fully rigid beam is assumed at point B which represent the hogging moment of the beam at the support under distributed load (when ϕ =0 , M = $wL^2/12$).

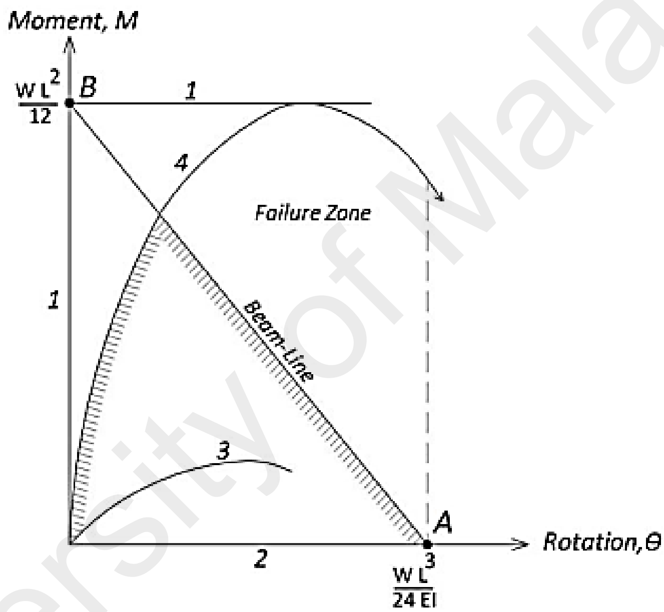


Figure 2.22: Moment-rotation characteristic of beam column connections (Elliott, 2002)

The line that connects Point A and Point is called "beam-line". In order to assess any connection, the moment-rotation plot needs to be verified against the beam-line. Thus, the detail descriptions of beam line are as below:

Line 1 : represents the behaviour of a perfectly fully rigid connection

Line 2 : represents the behaviour of an ideally pinned connection

Line 3 : If a moment-rotation curve (Line 3) fails to across the beam-line AB, the connection is considered as pinned due to the lack of the exhibited ductility

Line 4 : If a moment-rotation curve (Line 4) crosses the beam-line, the connection will have sufficient ductility and achieved required strength to be considered as a semi-rigid connection, and might be considered as a fully rigid connection.

Besides that, moment rotation curve incorporated with the beam-line is also used to determine the allowable moment capacity of a connection (M_E) and secant stiffness (S_E) (see Figure 2.23). The points along the beam-line define the relationship between the end moment and end rotation of the beam. M_E is defined from the intersection point of beam-line and moment rotation line. S_E also can be measured from this intersection.

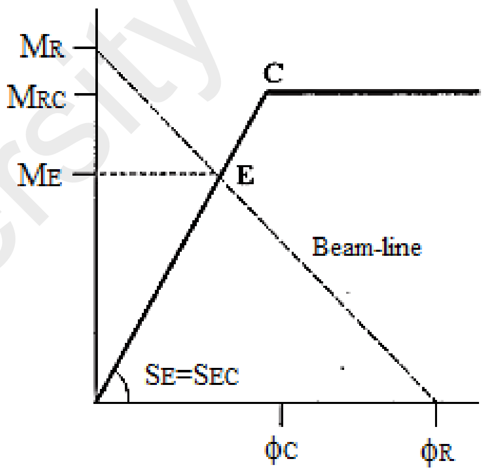


Figure 2.23: Intersection of moment-rotation line with beam line (Elliott *et al*, 2003)

Then, the value of stiffness factor, K_S also can be determined using calculation. The equations for S_E and K_S are given by equations (1) and (2).

$$S_E = \frac{M_E}{\phi_C} \tag{1}$$

$$K_S = {S_E}/{\left(\frac{4EI}{L}\right)} \qquad (2)$$

2.5.4 Connection Classification

Connection classification is a classification system for pinned, semi-rigid and full rigid beam-column connections. This system is proposed after Ferreira *et al*. (2005) and this classification consist five distinct zones, shown in Figure 2.24.

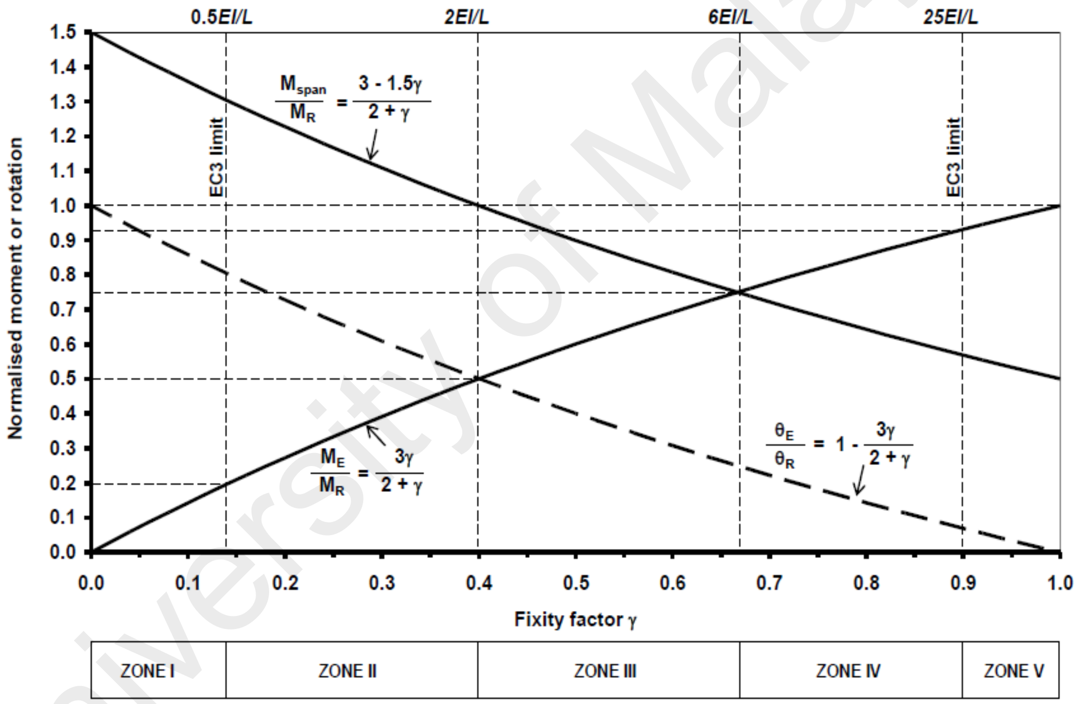


Figure 2.24: Connection classification system for pinned, semi-rigid and fully rigid beam to column connection. (Elliott & Jolly, 2013)

The descriptions of classification zones are as below:

Zone I :	$\gamma \leq 0.14$	Pinned-connections
Zone II :	$0.14 < \gamma \leq 0.40$	Semi-rigid with low strength
Zone III :	$0.40 < \gamma \leq 0.67$	Semi-rigid with medium strength
Zone IV :	$0.67 < \gamma \leq 0.90$	Semi-rigid with high strength

Zone V : $\gamma > 0.90$ Rigid Connection

In order to use the system, the value of Monforton's Fixity Factor (γ) must be determined first. The Monforton's Fixity Factor is given by the equation below:

$$\gamma = \left(1 + \frac{3\mathrm{EI}}{\mathrm{S_E L}}\right)^{-1} \tag{3}$$

2.5.5 Failure Modes and Crack Patterns

Extensive research on beam to column connection has been carried out all over the world. These researches are undertaken to investigate the behaviour of connection under static loading and simulated seismic loading. According to Meinheit and Jirsa (1981), the first experiment tests on beam-column connections were carried out in United States by the Portland Cement Association in the early 1960's and the results were published seven (7) years later by Hanson and Corner (1967). From the experimentala test results, Hanson and Corner (1967) have concluded that when the shear strength of the beam to column connection is computed using equations developed for reinforced concrete beams, a satisfactory estimate of the response of the beam-column connection under repeated load could be obtained .

Then, Meinheit and Jirsa (1981) also stress out about five possible failure modes that might occurred within the beam to column connection region. The possible mode of failure is shown in Figure 2.25.

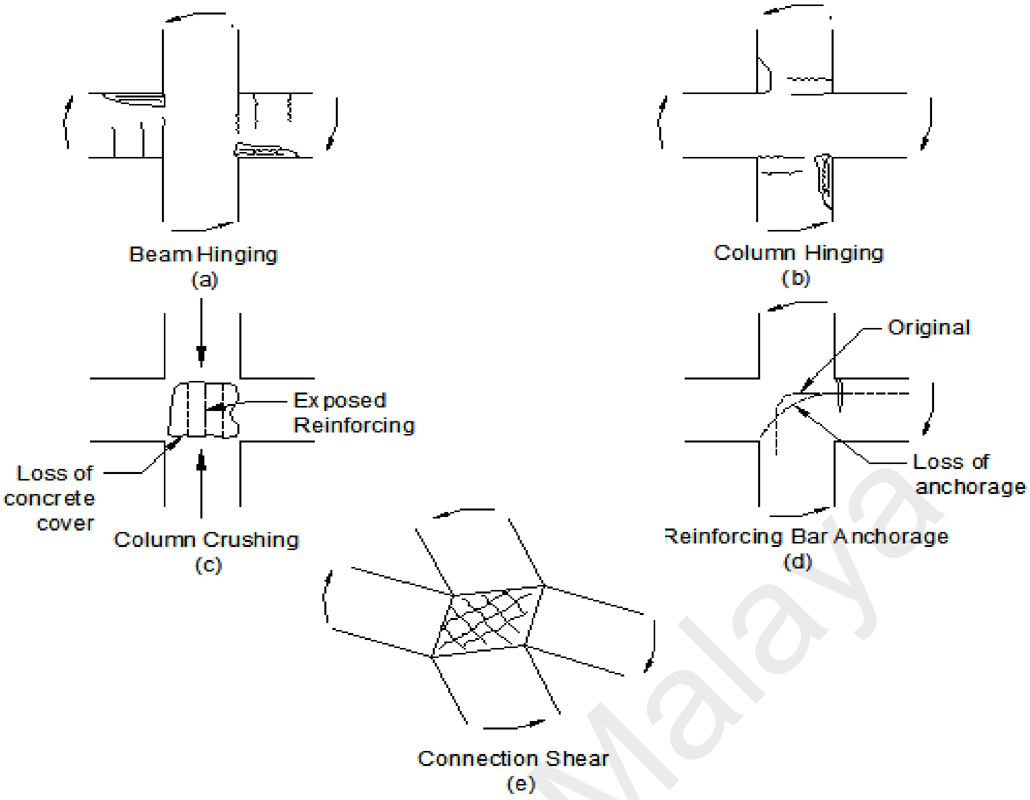


Figure 2.25: Possible failure modes within beam to column connection's region (Meinheit and Jirsa,1981)

The descriptions of the modes of failure are as below:

i. Beam hinging (Figure 2.25 (a)). This is the most desirable failure modes among others and it is a ductile flexural failure of the beam at the connection. Formation of hinges in the beams outside the connection allows for absorption of energy through large inelastic deformation without lost of strength. The mechanism is the same as beam hinging.

ii. Column hinging (Figure 2.25 (b)). Column hinging failure is less desirable than beam hinging. The frame may have a residual sway deflection and may be difficult to repair when the column hinged occurred.

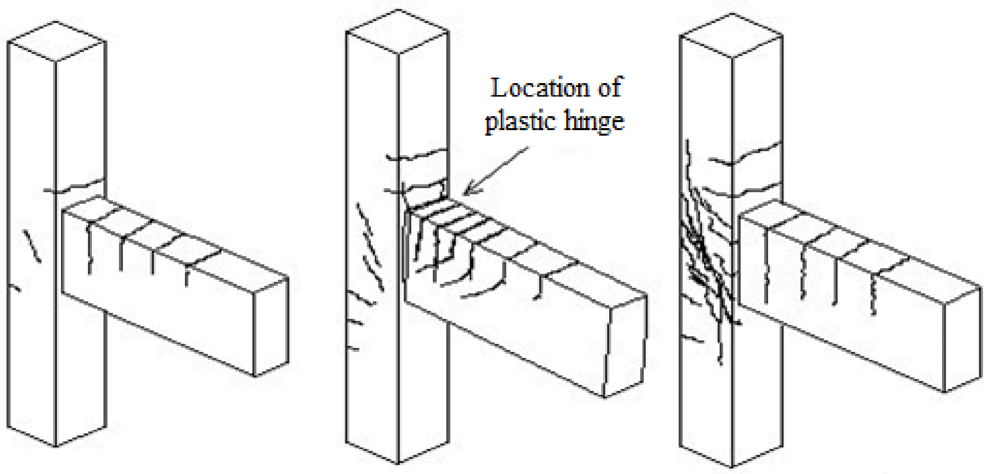
iii. Column crushing (Figure 2.25 (c)). This type of failure is undesirable since it affect the compressive load capacity of the column. The column compressive load capacity may be reduced under this condition especially in tied columns.

iv. Reinforcing bar anchorage (Figure 2.25 (d)). It is undesirable failure modes if the loss of anchorage of the reinforcement happened in exterior connections. This is because lateral shear can no longer be transmitted by the frame. This type of failure also causes a reduction in the energy absorbing ability of the structural system.

v. Connections shear (Figure2.25 (e)). The consequences of failure of the connection in shear are the same as loss of anchorage, an inability of the frame to transfer lateral shear and declining energy absorbing ability.

In 1999, Hamil and Scott also have done research on beam to column connection. From that research, three types of failure modes within beam to column connection zone were obtained. The failure modes are shown in Figure 2.26. Again, according to Hamil and Scott (1999), all specimens exhibited flexural cracking in the beam and the column regions followed by diagonal cracking in the connection itself, as shown in Figure 2.26(a). Further shears was then carried by the concrete struts between the cracks assisted by confinement provided by the connection zone ties.

Then, the specimens were introduced to increment loading and failure occurred. There were two different failure mechanisms. Those failures are if the ultimate moment of resistance of the beams was reached, then a plastic hinged formed in the beam at face of the column, as shown in Figure 2.26(b). If excessive shear cracking developed in the connection zone, before the beam reached its ultimate moment, then an extensive joint cracking failure occurred. This is shown in Figure 2.26(c).



(a) Flexural cracking in the beam and column region (b) Beam plastic (c) Extensive joint

Figure 2.26: The failure modes obtained from experiment (Hamil & Scott, 1999)

2.6 Analytical Model

Analytical model is one of the approaches that can be taken to determine the behaviour of beam column connection instead of empirical, experimental, informational, numerical and mechanical methods. This method used the basic concepts of structural analysis which are equilibrium, compatibility and material constitutive relations in order to obtain the rotational stiffness and moment resistance of a connection due to its geometric and mechanical properties (Diaz *et al.* 2011).

Currently, limited studies are available on the analytical equations to predict the semirigid connection behaviour. Recent proposal was made by Ferreira and Elliott (2002) suggested that the important parameters in determining the connection behaviour are moment resistance, rotation and stiffness. Flexural strength and rotational stiffness must meet simultaneously as the requirement to this analytical equation prediction. This is also discussed in Elliott, *et al.* (2003), Ferreira and Elliott (2002) and Elliott *et al.* (2004).

In order to predict the semi-rigid behaviour, the rotational stiffness (S) is defined as:

$$S = \frac{M_{RC}}{\phi_C} \tag{4}$$

Where: M_{RC} = moment resistance of the connection and

ϕ_c = is the total end relative rotation due to M_{RC}.

In order to obtain the moment resistance, M_{RC} of the connection, a rectangular stress block approach according to BS8110 is adopted:

$$M_{RC} = zf_y A_S d \tag{5}$$

Where: z = lever arm of the connection

f_y = tensile strength for the tension bars

A_s = steel bars area

d = effective depth

Total end relative rotation ϕ_c is obtained from these two deformations which are:

i. Joint opening at the interface joint opening at the interface.

It is due to elongation of top reinforcement bar (see Figure 2.27)

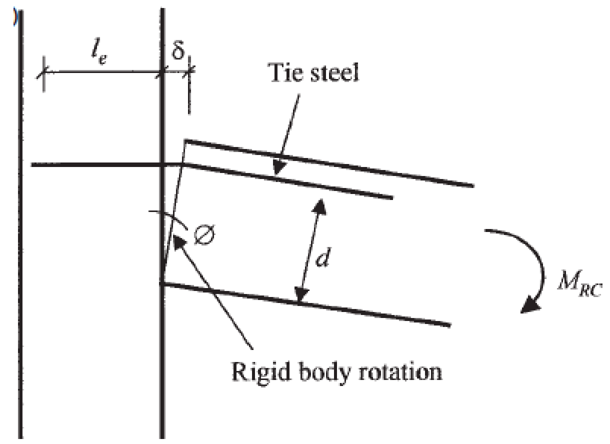


Figure 2.27: Interface joint rotation due to joint opening (Elliott *et al*, 2003)

The elongation of top reinforcement bar is define as :

$$\phi_C = \frac{\delta}{d} \tag{6}$$

The deformation g is equal to yield strength in the reinforcement bar times embedment length

$$\delta = \frac{f_y}{E_s} \text{ x } l_e \tag{7}$$

Where :

l_e : Embedment length of reinforcement across column (see Figure 2.28). l_e is taken as lesser of a length over which the stress distribution along the bar uniform

E_S : Modulous's Young of steel

Then,

$$\phi_C = \frac{f_y l_e}{E_S d} \tag{8}$$

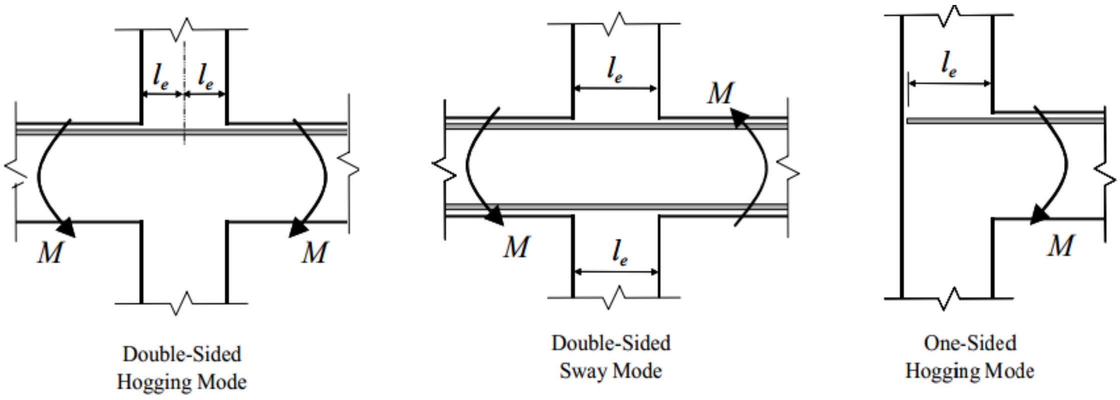


Figure 2.28: Embedment length of reinforcement across columns (Elliott *et al*, 2004)

ii. Beam end rotational deformation

It is due to the curvature of the beam in a region where the curvature and tensile stress in the top bars in the beam are found to be constant (see Figure 2.29). There is a concentration of crack causing curvature that is constant within plastic hinge length l_p. The l_p depends on the load path from center of rotation and the type of connector bearing and whether the force is transmitted to the beam by a cast in steel plate or by suspensions, see Figure 2.29.Thus,

$$\phi_C = \frac{M_{RC} \text{ x } l_p}{E_C \text{ x } I_{beam}} \tag{9}$$

So, total end relative rotation,

$$\phi_C = \left(\frac{f_y l_e}{E_S d}\right) + \left(\frac{M_{RC} \text{ x } l_p}{E_C \text{ x } I_{beam}}\right) \tag{10}$$

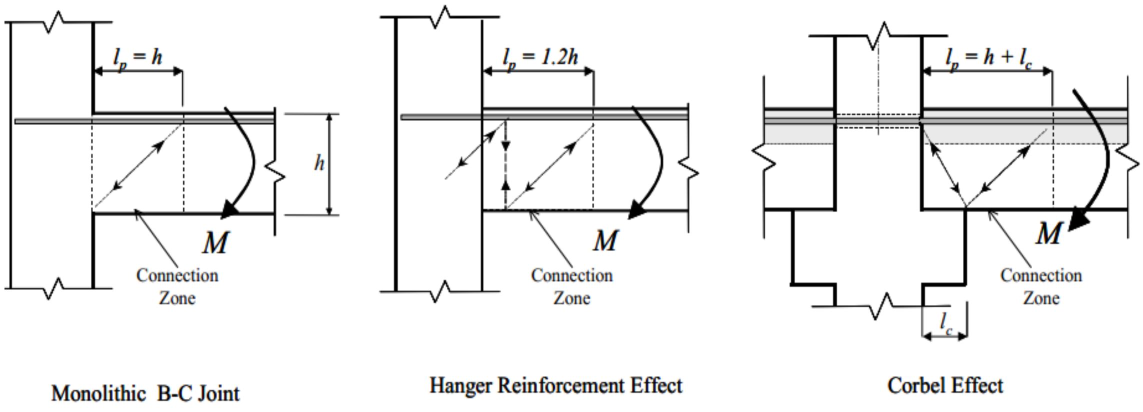


Figure 2.29: Plastic hinge length for types of precast connections (Elliott *et al*, 2004)

The required moment capacity M_{ER} and the allowable design moment M_{ED} for the connector at ultimate limit state (ULS) can be obtained from the intersection of S with the beam-line and the expressions are as below:

$$\frac{M_{ER}}{M_R} = \frac{M_{ED}}{M_d} = \left(1 + \left(\frac{2E_C I_{CR}}{L}\right)\left(\frac{\phi_C}{M_R}\right)\right)^{-1} \tag{11}$$

Thus, by substituting Eq. (5) for M_{RC} and Eq. (10) for ϕ_c, Eq. (11) is rewritten as:

$$\frac{M_{ER}}{M_R} = \frac{M_{ED}}{M_d} = \left(\left(\frac{L + 2l_P}{L}\right) + \left(\frac{2E_C I_{CR}}{E_S A_S dz}\right)\left(\frac{l_e}{L}\right)\right)^{-1} \tag{12}$$

CHAPTER 3: RESEARCH METHODOLOGY

3.1 Introduction

This study involved laboratory testing of a total of three (3) specimens. These specimens are having the same geometric and material properties. Repetitive testing is done to confirm the result. In order to verify the results, validation with analytical method is made.

The steps taken to accomplish this study are divided in three (3) stages which are design stage, experimental works stage and result analysis stage. The details about the stages are described below:

i. Design stage

   At this stage, an existing precast beam to column connection was selected. Then, a new proposed precast beam to column connection was designed and modified based on the existing precast connection. This proposed connection is designed to fulfill such requirements which are able to resist moment resistance, easy in constructing, fast in erection and cost effecive. Architectural demand also is taken into account where no corbel used for this connection. Architectures find it hard using corbel due to its limitation.

ii. Experimental works stage

   This stage involves the preparation for laboratory testing for proposed precast beam to column connection. The works involves are specimens fabrication at site, assemblage specimen components at laboratory and specimens testing.

iii. Result analysis stage

At this stage, the results obtained from the experiment are analysed. The analysis of result involve determining the moment resistance of the connection, plotting the graph for moment rotation (M-ϕ) relationship, load displacement relationship and beam line. Then, types of connection for proposed precast beam to column connection is determined using connection classification system according to Monforton's Fixity Factor. Validation testing result also made with analytical approach.

To simplified the methods, the flowchart below shows the chronology of the methods (Figure 3.1).

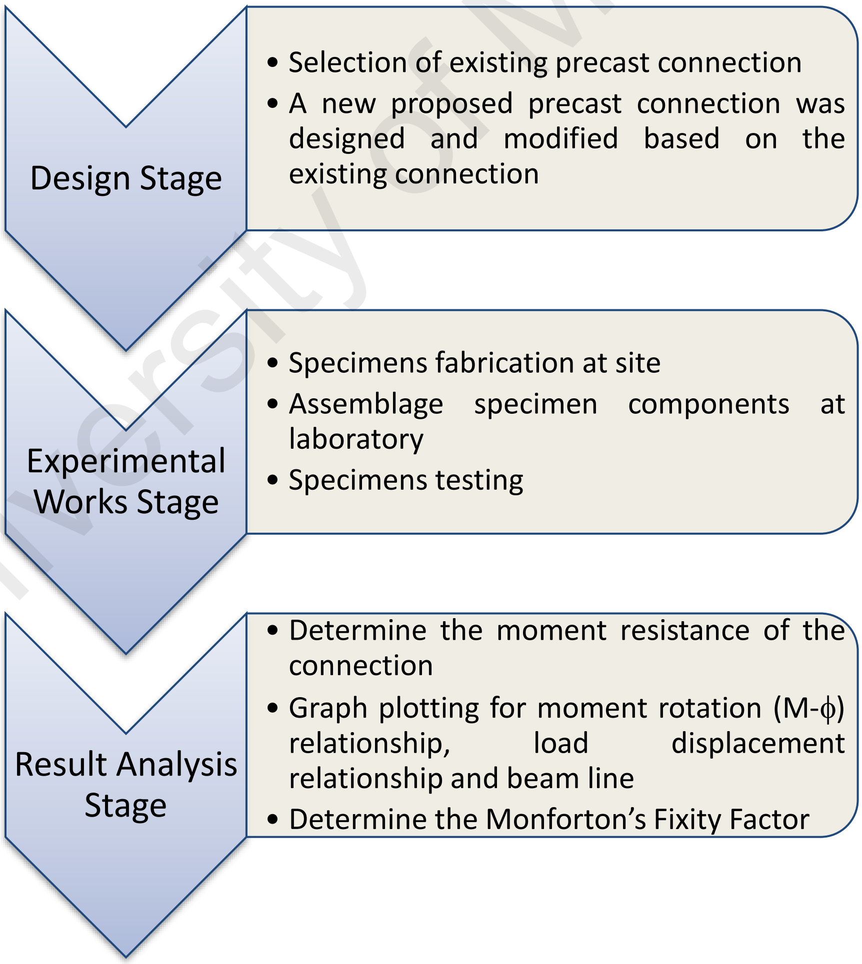


Figure 3.1: The flowchart of the methods

3.2 Design Stage

This study started with proposal of precast beam to column connection. As mentioned earlier, an existing precast beam to column connection was selected. Then, a relevant precast connection was designed. This connection was designed based on recommendation of BS8110:1997. This connection is called Billet Connection and in this testing, the specimens were labelled as BIC 1, BIC 2 and BIC 3. The descriptions of the connection are described in Section 3.2.1.

3.2.1 Description of the Connection

In this study, a new precast beam to column connection has been proposed and it is hidden corbel connection. This connection using beam half joint and cast in steel insert in the column (billet) and both components were jointed together. This connection is modification of existing precast connection proposed by previous researcher. The existing precast beam to column connection is shown in Figure 3.2.

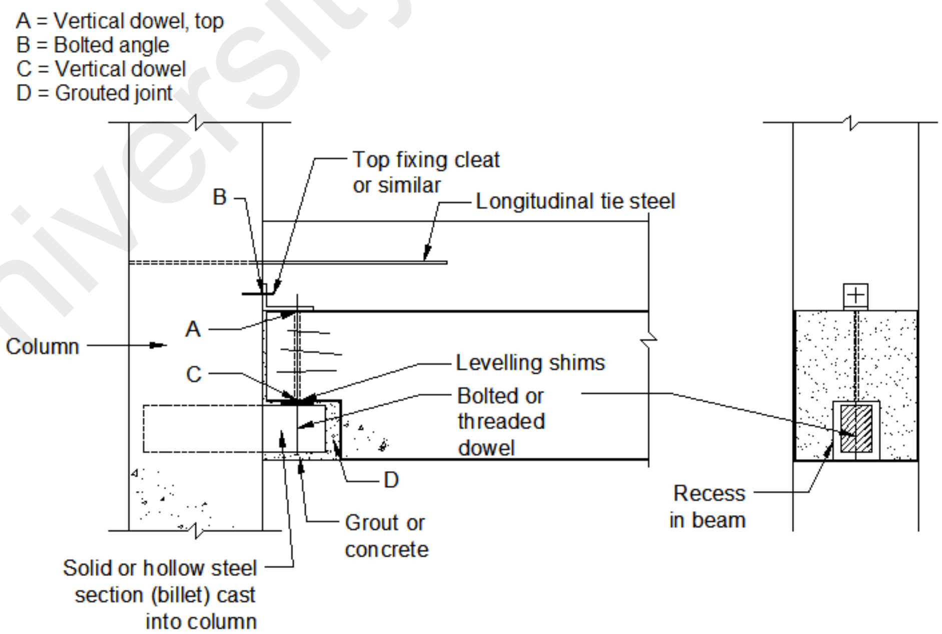


Figure 3.2: Existing precast beam to column connection (Fib, 2008)

The modifications that have been made to the existing precast connection are the location of additional top reinforcement bar to a certain level within the beam. Since the top reinforcements are within the beam, only small amount of grout need to be used instead of a lot of concrete topping for existing precast connection. Besides, the proposed connection used the mechanical connector within the column to connect the additional top reinforcement bar to the column. The proposed connection and its detailing are shown in Figures 3.3, Figure 3.4 and Figure 3.5.

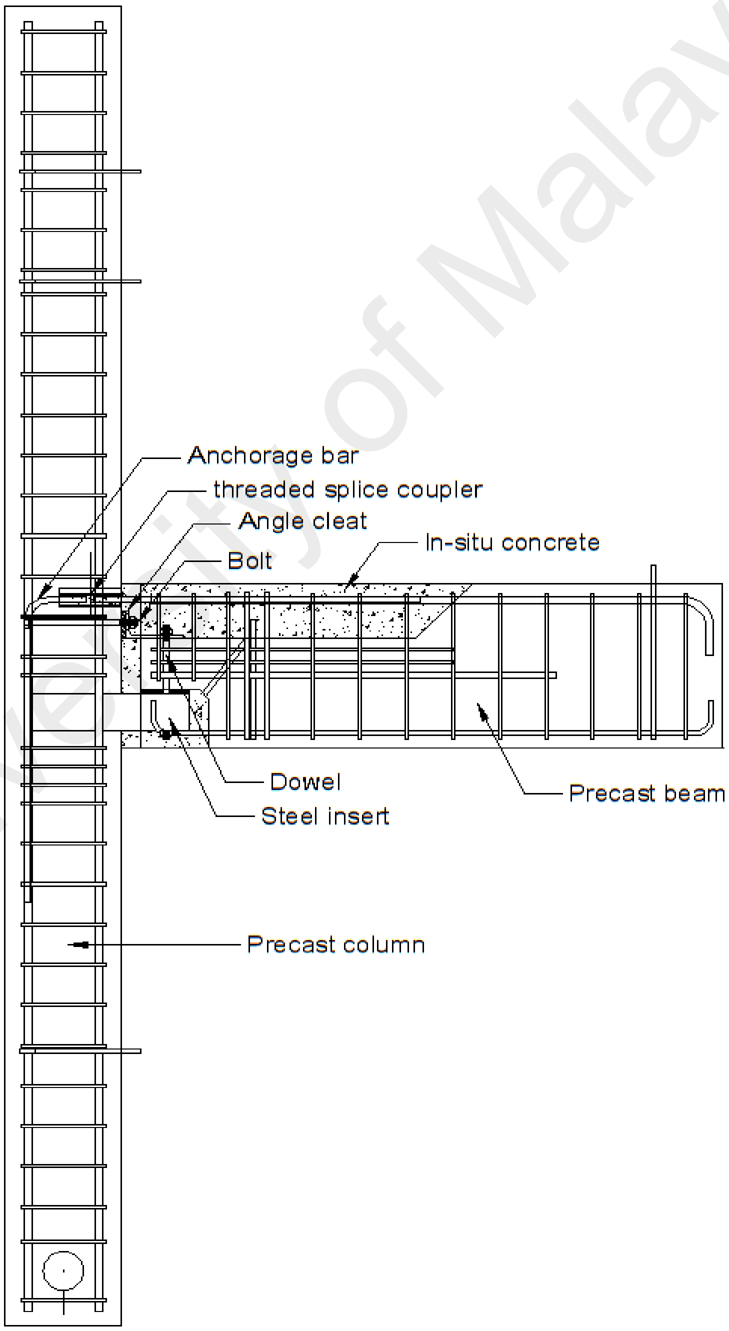


Figure 3.3: The proposed precast beam to column connection

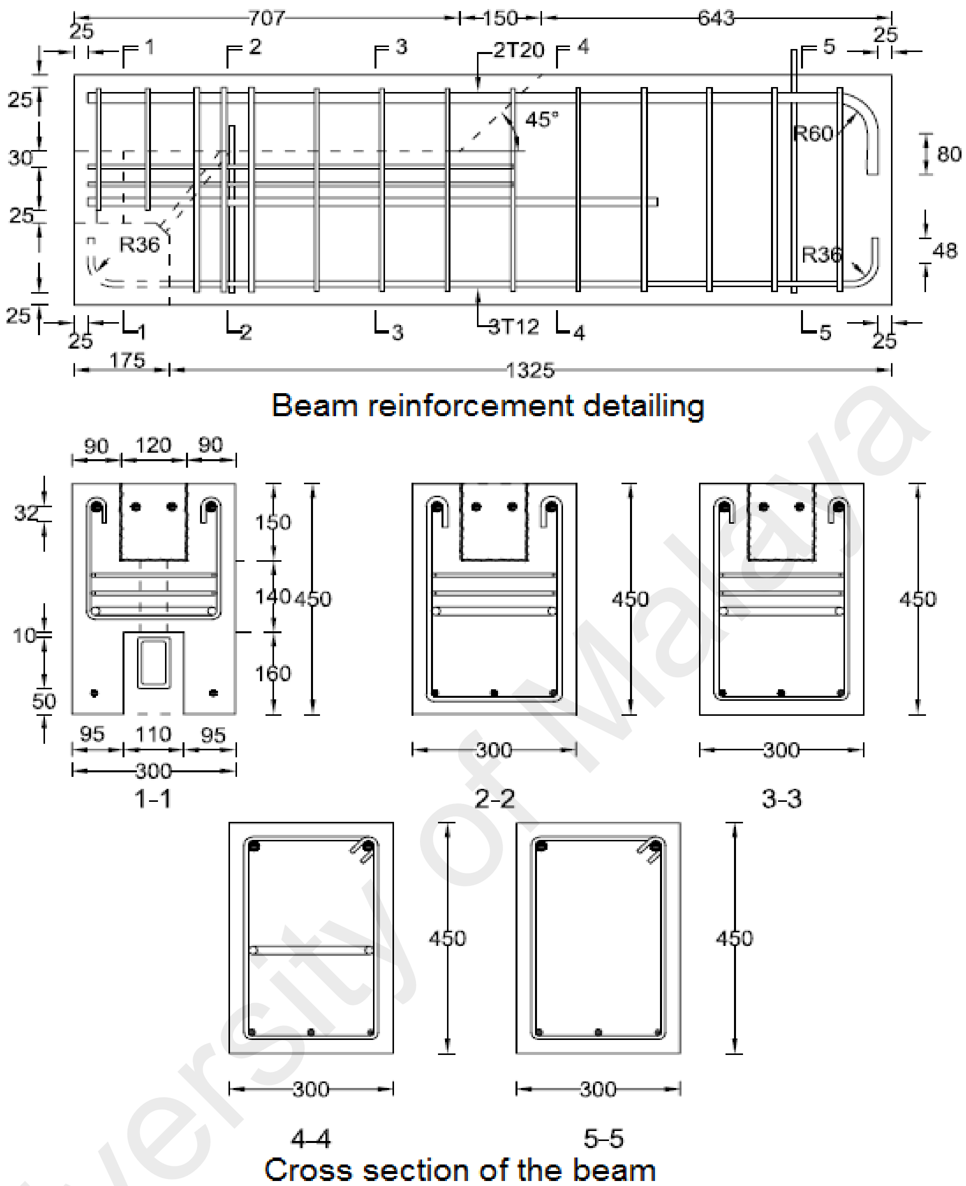


Figure 3.4: Precast beam half joint detaling

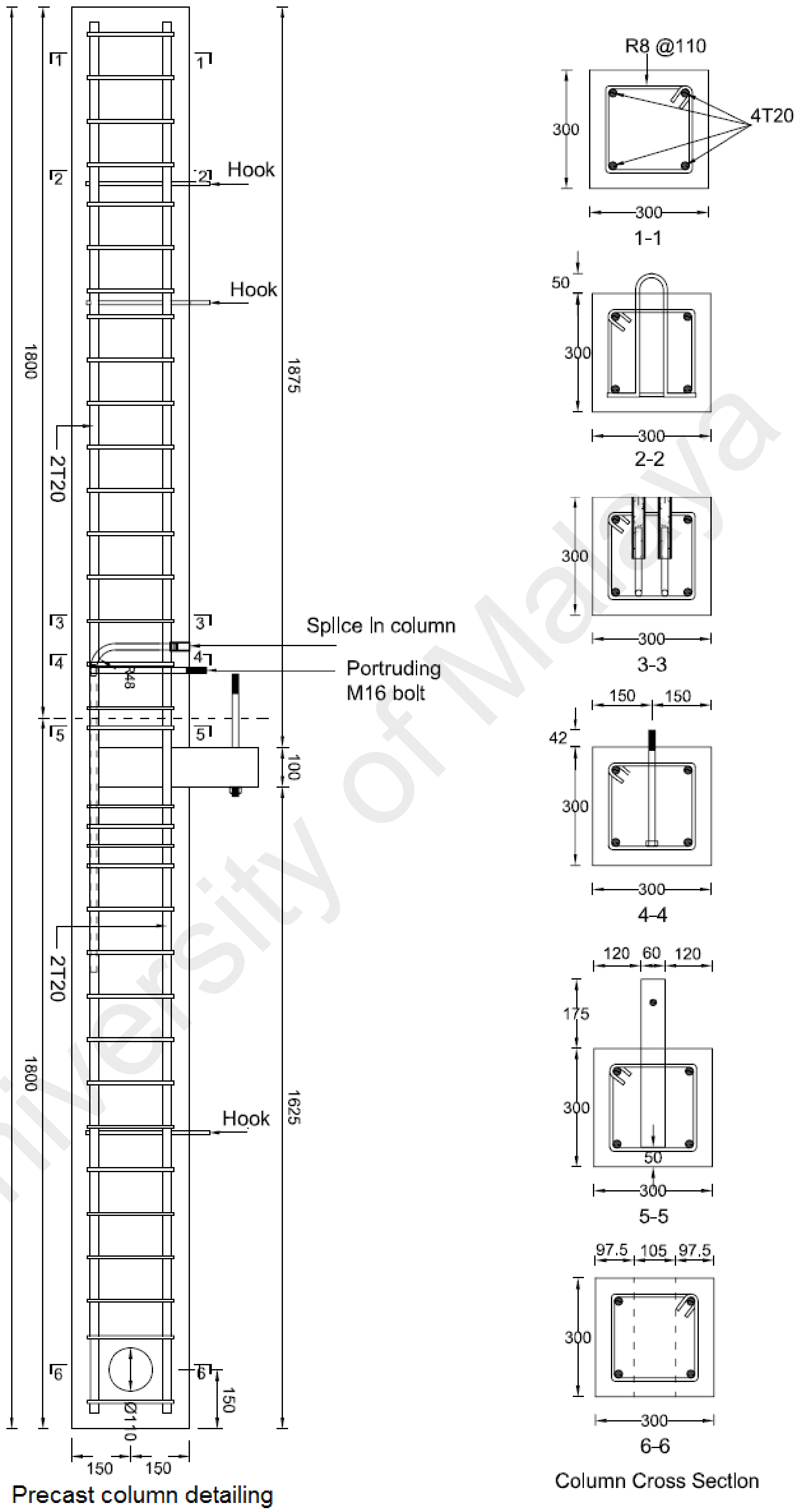


Figure 3.5: Precast column detailing

Basically, this connection was designed based on the recommendation of BS8110:1997. This connection consists of steel insert with dowel to support the beam. The steel insert is rectangular hollow section (RHS) of size 100 x 60 x 8mm with infill concrete to make it stiffer. The dowel of 16mm diameter is connected between the steel insert and the angle cleat. The cleat is then bolted to the column. The beam size used for this connection is 300 x 450 mm and 300 x 300 mm for the column size. The top of the beam has a recess of 150 mm deep and 860 mm length to permit the hand placement of two (2) no. T16 mm rebar x 770 mm length anchored to the column using threaded splice couplers. The tensile strength of the couplers must be greater than tension bar which is 460 N/mm^2. The concrete grade used for this specimen is 40 N/mm^2 whilst 460 N/mm^2 steel strength is used for the main bars (high tensile steel) and 250 N/mm^2 steel strength for links and stirrups. All design calculations are shown in Appendix A.

3.3 Experimental Work

3.3.1 Fabrication at Site

All sample components (precast beam and precast column) were fabricated and casting at Teraju Precast Sdn. Bhd at Dengkil, Selangor. All components were cast using ready mix concrete with grade 40 concrete. The compressive strength test was conducted by Teraju Precast Sdn. Bhd. at 28 days and all the concrete achieved the required strength. Inspection was made to ensure the reinforcement and stirrup follow as per drawing. Figure 3.6 to Figure 3.8 shows the construction work at site.

Figure 3.6: Tying the reinforcement

Figure 3.7: Reinforcement inspection

Figure 3.8: Reinforcement caging ready to be put in to the mould

3.3.2 Sub Assemblage of Specimen Components at Laboratory

All assembly works for specimen's components were done at Construction Research Institute of Malaysia (CREAM)'s laboratory. The assembly works involved stages such as erection of precast connection, preparation of formwork, grouting process, casting of concrete at jointing part, curing process and finally painting process.

For assembling of precast column and precast beam, the precast column was lifted using gantry crane and was placed into column support which restrained to the strong floor (see Figure 3.9a). Then, the precast beam half joint was placed on the billet projecting from the column face (column connection). Whilst at the other side of the beam were seated on the temporary support.

The gap between beam and column was filled with grout and simple timber formwork was formed (Figure 3.9b and 3.9c).

b) Simple timber formwork for grouting

a) Assembling of precast column and precast beam

c) Close-up bottom formwork

Figure 3.9: Erection of connection

For grouting, Sika grout 215 with the strength of 50 N/mm^2 at seven (7) days for flowable mixing and 65N/mm^2 at seven (7) days for pourable mixing was used to fill the dowel holes in precast beam. For this connection, pourable mixing was selected. The mixing proportion for pourable grout is shown in Table 3.1.

Table 3.1: Pourable grout mixing proportion

Sika grout 215 (kg)	1.90	25
Water (litre)	0.30	4.0
Volume mortar (litre)	1.00	13.2

Interpolation was made in order to determine volume of water and grout needed. The grout was mixed using hand mixer (Figure 3.10a). Then the grout is filled into grout mould for cube test purposed (Figure 3.10b) and into the gap within dowel holes in precast beam (Figure 3.10c and Figure 3.10d)

a) Grout mix

b) Grout mould for cube test

c) Before grouting

d) After grouting

Figure 3.10: Grouting process

For concreting works, concreting only involved the small recess of 150mm deep and 860mm length, at the top of the beam. The concrete mix design is attached in Appendix B. This area was concrete after two (2) no. T16 mm rebar x 770 mm length is placed and anchored to the column using threaded splice couplers. The concreting work was done at CREAM's laboratory (Figure 3.11). Slump test was done in order to determine the workability of the concrete (Figure 3.12).

Figure 3.11: Concrete mixing at laboratory

Figure 3.12: Slump test to determine the workability of concrete mixing

Total (8) eight test cubes for each concrete batch was prepared using 150mm mould. The concrete was filled into mould with 3 (three) equal layers and must be fully compact to reduce air trapped that might cause low concrete strength. Vibration table was used to compact the concrete (Figure 3.13).

Figure 3.13: Concrete in moulds for cube test

At beam-column jointing part, a handheld concrete vibrator was used to compact the concrete (Figure 3.14).

Figure 3.14: Concreting at jointing part

Last work for specimen preparation was the painting process (Figure 3.15). The whole specimen was paint with white colour wheareas this colour is suitable to be used in observed the cracks and cracks marking.

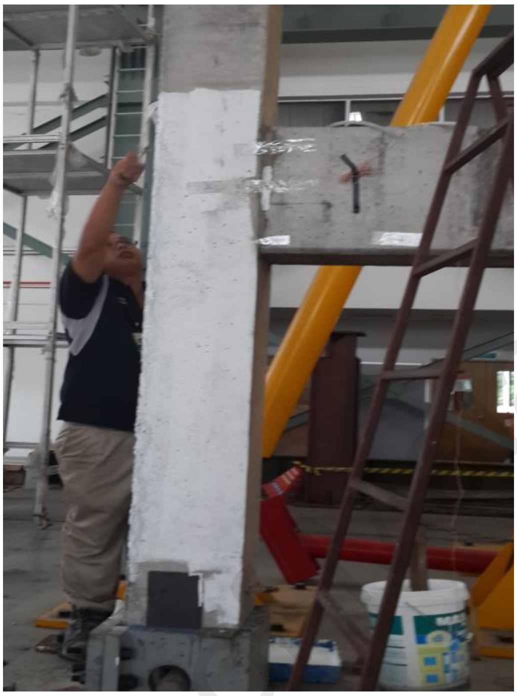


Figure 3.15: Painting process of the whole specimen

3.3.3 Experimental Setup and Instrumentation

The setup for the testing is shown in Figure 3.16. The precast column was restrained at both the top and bottom ends of the column. The top column was tied back to the steel frame that was anchored to the strong floor while bottom column is restrained using column support. This setup was designed to ensure there is no rotation for the column while loading is applied to the beam. Reversible load is applied at a distance of 3d (where d is effective depth of the beam) from the column face. This action will produce moment at connection until failure.

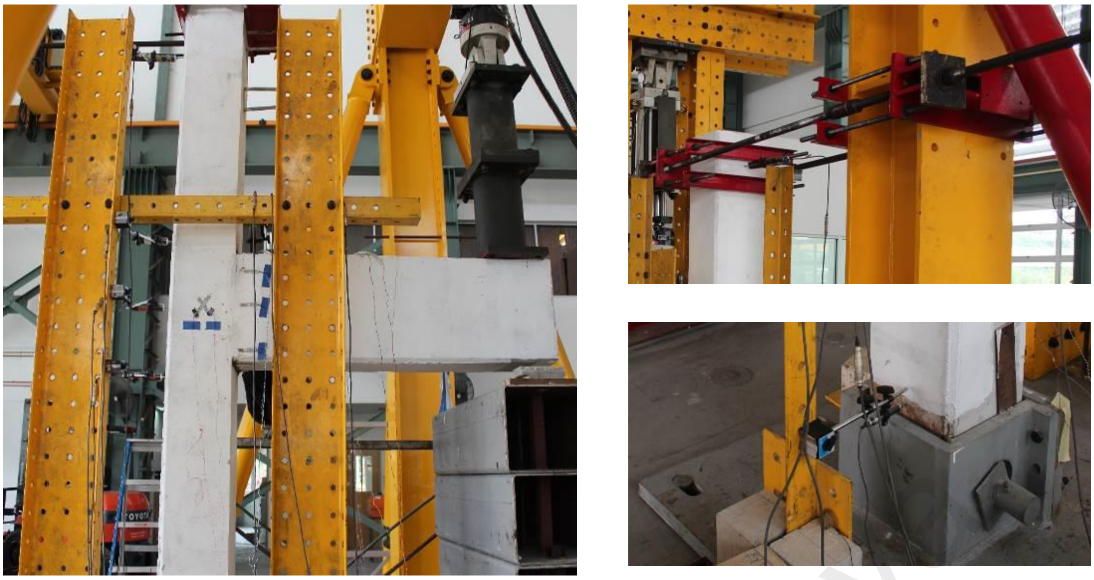


Figure 3.16: Experimental setup for flexural test

For instrumentation, LVDT (Linear Variable Displacement Transducer) was used to capture displacement data from the experiment and total eight (8) numbers of LVDT were used to measure displacements of precast beam and column. To record the displacement values produces from LVDT, a data logger was used (Figure 3.17). Whilst to measure the strain in steel reinforcement and concrete, three (3) types of strain gauges is used which were steel strain gauge, concrete strain gauge and concrete rosette strain gauge. The strain gauges used are shown in Figure 3.18 and Figure 3.19. The overall instrumentation of this testing is shown in Figure 3.20 and Figure 3.21.

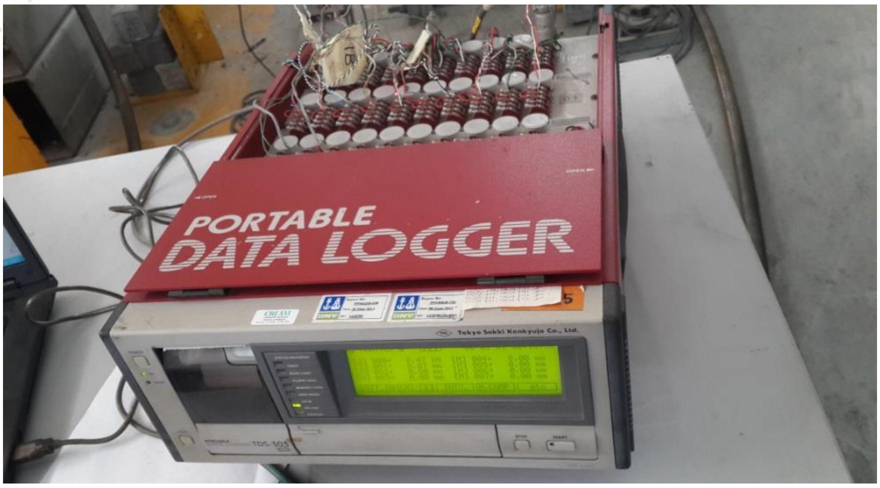


Figure 3.17: Data logger that used to capture data from LVDT and strain gauge

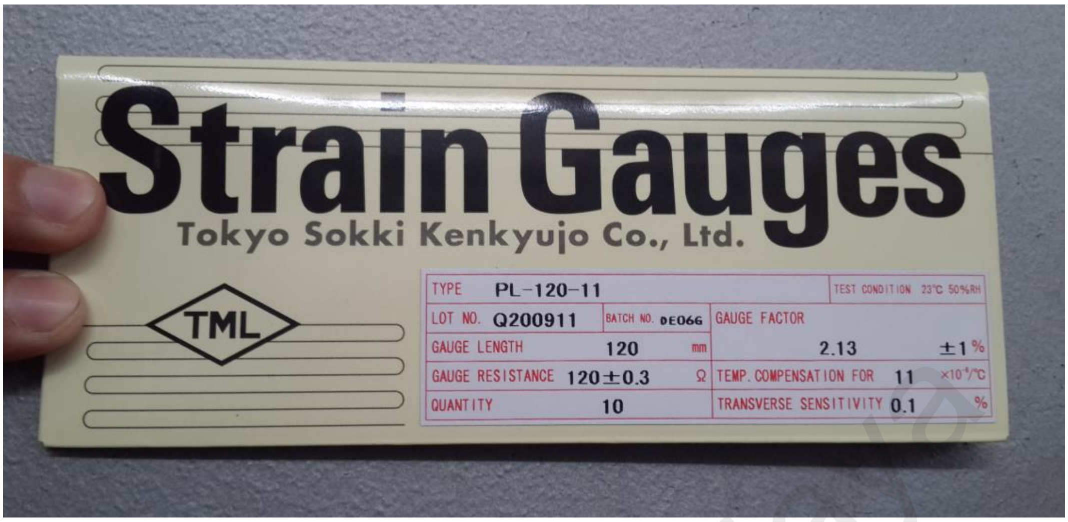


Figure 3.18 : Concrete strain gauge

Figure 3.19: Steel strain gauge

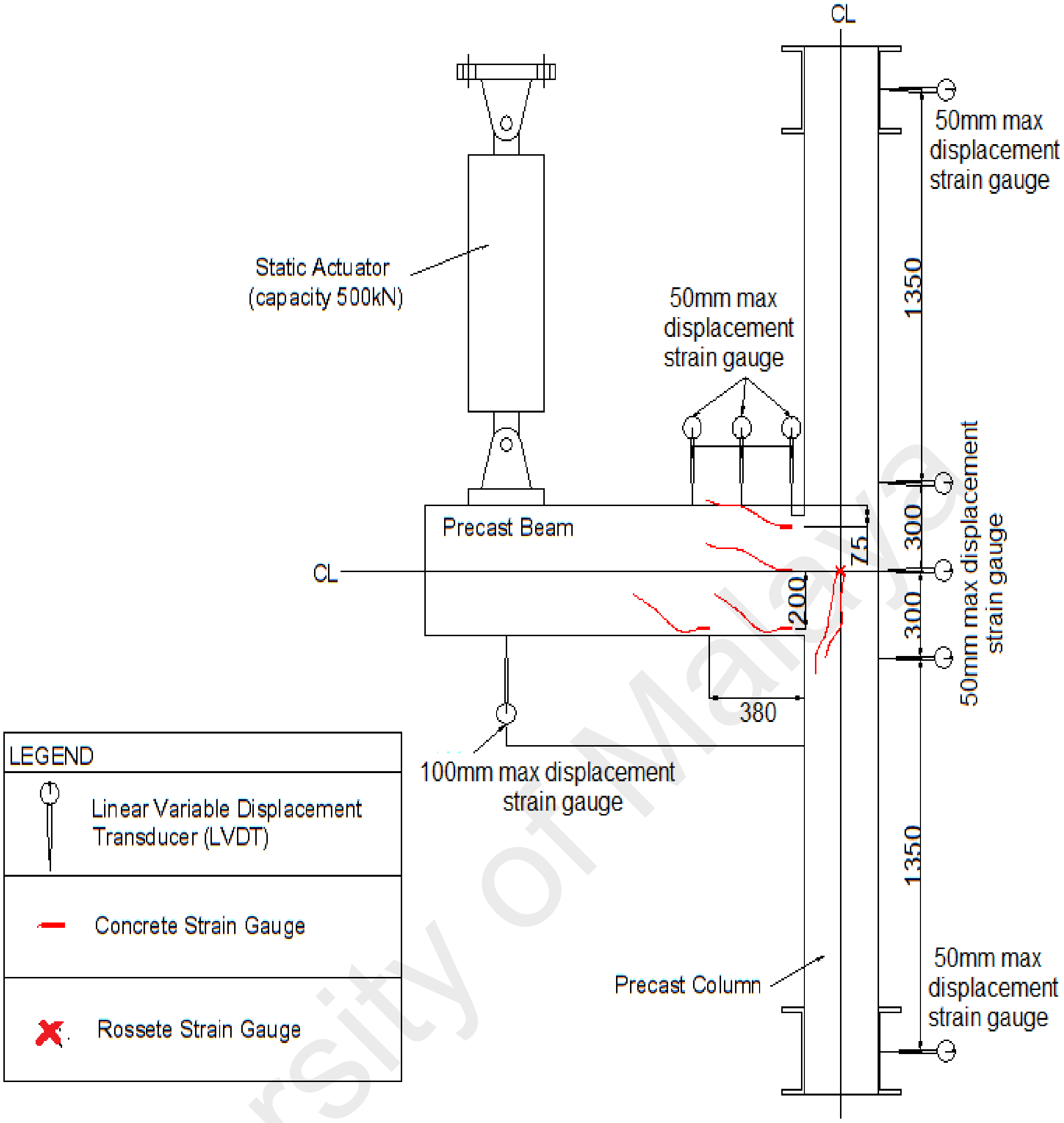


Figure 3.20: Location of LVDT and concrete strain gauges

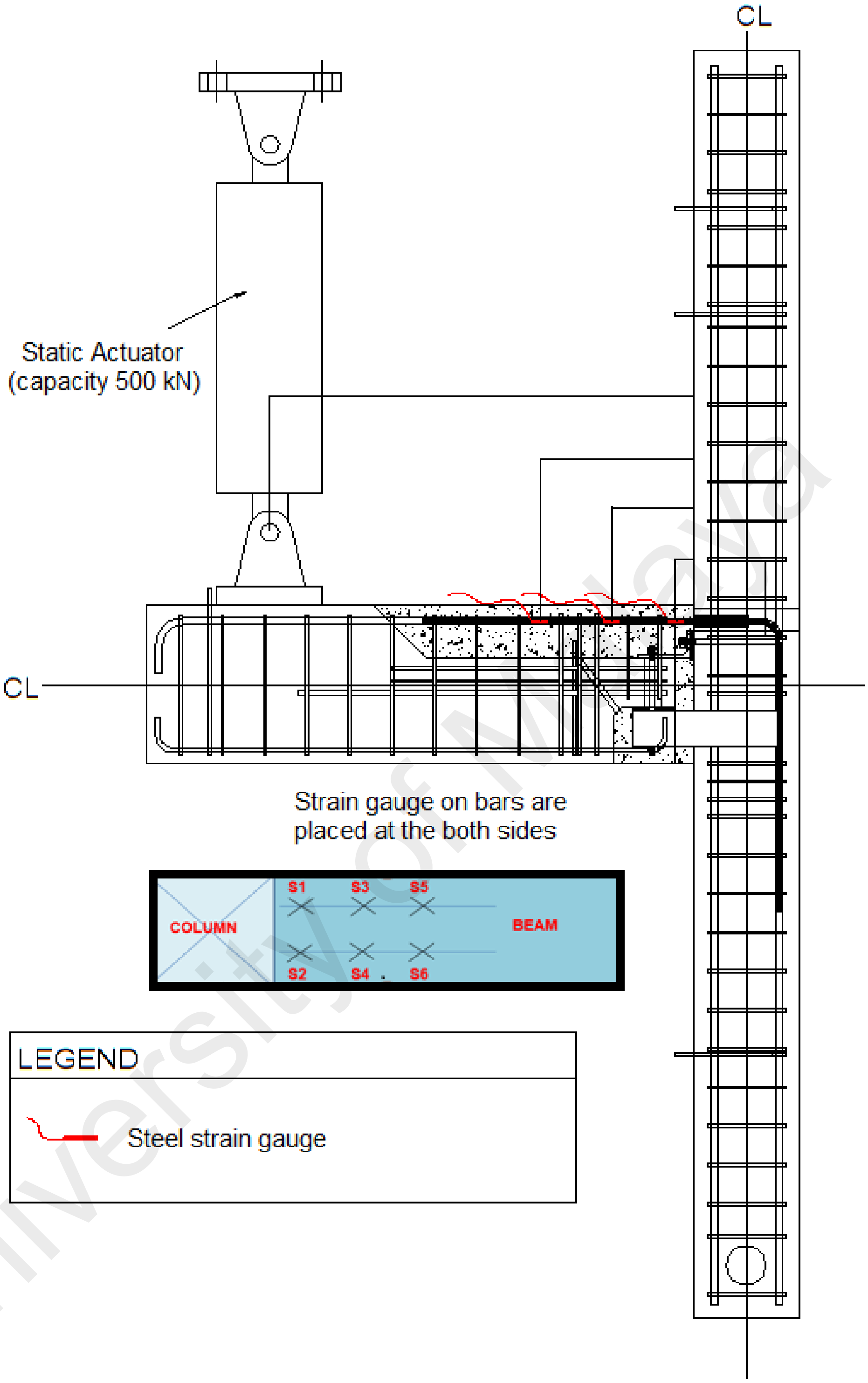


Figure 3.21: Locations of steel strain gauges

3.3.4 Testing Procedure

The testing on connections was conducted at seven (7) days after concrete strength reached 40±5 N/mm^2. In order to study the stiffness of the connections, the bending load (P) was applied in six (6) reversible cycles followed by monotonic loading up to failure. The load was applied on precast beam at distance of 3d from the column face (which is Lp =1350mm) and with the increment of 5kN. The load was reversed at first crack loading, second crack loading and then an increasing load until the connections were not capable of supporting any further bending moment.

3.4 Analytical Method

3.4.1 Moment-Rotation (M-ϕ) Calculation Technique

In order to plot the graph moment-rotation relationship, the values of moment, rotation and stiffness need to be calculated as discussed in Section 3.4.1.1, Section 3.4.1.2 and Section 3.4.1.3 respectively.

3.4.1.1 Calculation of Moment

The moment, M was calculated by multiplying the applied load, P (recorded by actuator) with the contraflexure length which is at the distance of 3d (1350mm) from the face of the column and also the moment . This value was then added to the initial bending moment of the selfweight of the beam. All of these are illustrated in Figure 3.22. The selfweight of beam is 4.86 kN (0.30m x 0.45m x 1.5m x 24 kN/m^3). The calculation of moment is as below:

$$\text{Moment, } M = [1.35P + 0.634\ sw]\ \text{kNm}$$

$$= [1.35P + 0.634\ (4.86)]\ \text{kNm}$$

$=[1.35P + 3.081]$ kNm   P : unit in kN

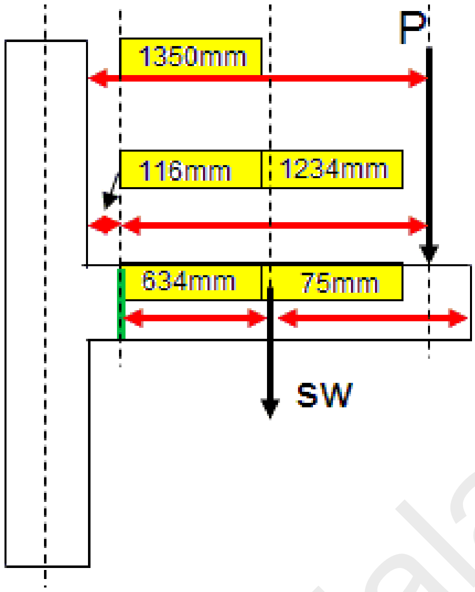


Figure 3.22: Typical details for moment calculation method

3.4.1.2 Calculation of Rotation

According to Hasan *et al.* (2011), there is no standard or normative method to experimentally measure the relative beam to column rotation and variety techniques have been used by various researchers. In this study, rotation (ϕ) was determined from Gorgun (1997). The rotation was determined from the relative vertical deflection of the compression face of the beam with reference to the column. This method was also used by Mahdi (1992) and Ferreira (1999). The connection rotation is determined from Eq. 13.

$$\text{Connection rotation } (\phi) = \text{column rotation } (\phi_{column}) - \text{beam rotation } (\phi_{beam}) \qquad (13)$$

The method to obtain beam rotation (ϕ_{beam}) and column rotation (ϕ_{column}) are as below:

i. Beam rotation

Referring to Figure 3.23, LVDTs are mounted along the beam and labelled as LVDT 6, LVDT 7 and LVDT 8 to measure the vertical deflection, δ.

The relative rotation was produced as follows:

$$\phi_{\text{LVDT 6}} = \frac{\text{LVDT 6}}{\text{x}} \tag{14}$$

$$\phi_{\text{LVDT 7}} = \frac{\text{LVDT 7}}{\text{y}} \tag{15}$$

$$\phi_{\text{LVDT 8}} = \frac{\text{LVDT 8}}{z} \tag{16}$$

Thus, relative rotation, $\phi = \phi_{\text{LVDT 8}} - \phi_{\text{LVDT 6}} = \frac{\text{LVDT 8} - \text{LVDT 6}}{\text{z} - \text{x}}$ (17)

The relative rotation was produced by dividing relative deflection with relative distance of LVDT 8 and LVDT 6.

ii. Column rotation

The same method applies for column rotation. The LVDT 1, LVDT 2, LVDT 3, LVDT 4 and LVDT 5 were placed horizontally at the column. However, to calculate the column rotation, only readings from LVDT 2 and LVDT 4 were taken into account. Readings from LVDT 1 and LVDT 5 were adopted to monitor any rotation at top and bottom column's support. The calculation of column rotation is as follows:

$$\phi_{\text{LVDT 2}} = \frac{\text{LVDT 2}}{\text{a}} \tag{18}$$

$$\phi_{\mathrm{LVDT\ 4}} = \frac{\mathrm{LVDT\ 4}}{\mathrm{b}} \tag{19}$$

Thus, relative rotation, $\phi = \phi_{\mathrm{LVDT\ 2}} + \phi_{\mathrm{LVDT\ 4}} = \frac{\mathrm{LVDT\ 2+LVDT\ 4}}{\mathrm{a+b}}$ (20)

The relative rotation was produced by dividing column relative deflection with actual vertical distance of LVDT 2 and LVDT 4.

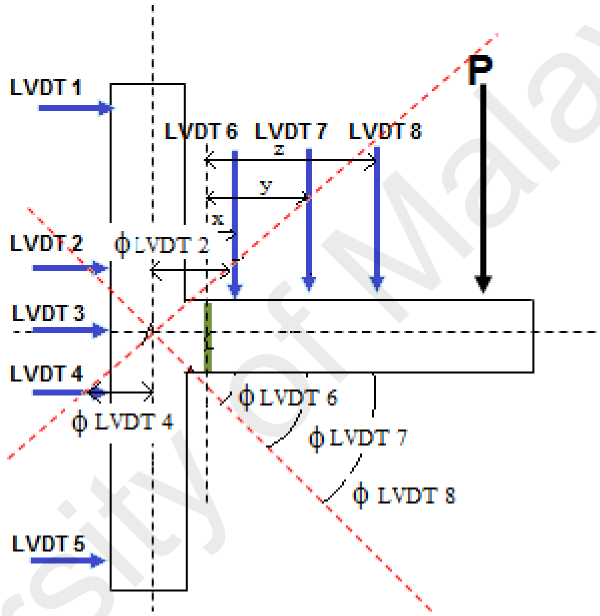


Figure 3.23 : Typical details for calculation of connection rotation

3.4.1.3 Calculation of Stiffness

The rotational stiffness (S), is calculated from the slope of the M-ϕ graph. In order to calculate the stiffness, equation (4) in Section 2.6 is adopted. The equation is:

$$S = \frac{M}{\phi} \tag{21}$$

3.4.2 Beam Line Method

In the moment-rotation (M-ϕ) graph, the beam line intersects the vertical axis at moment value which is the end moment of a fully fixed beam. Whilst at the horizontal axis, the beam line intersects at rotation value which is rotation at the end of a simply supported beam. In order to draw the beam-line, there is a need to calculate the end moment of the connection and the rotation. The end moment of connection is obtained from the equation M= F x z where this calculation involved the internal forces in the connection (Refer Figure 3.24). For detail calculation, please refer Appendix C1.

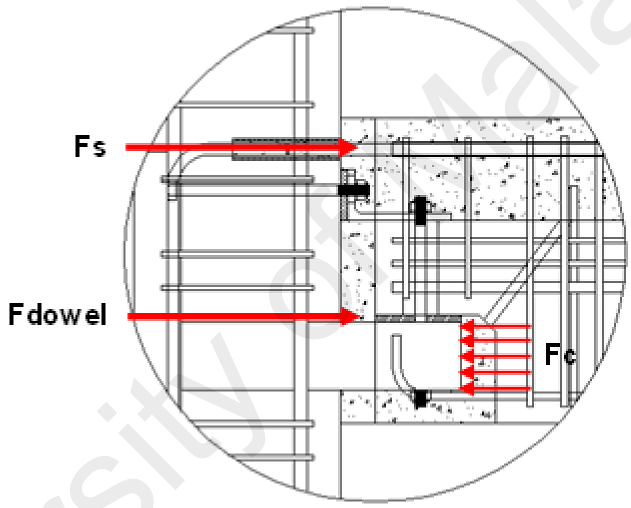


Figure 3.24: Internal force in the connection

The rotation can be calculated once the moment is obtained. This is based on linear interaction points of end moment and end rotation. This beam line having a gradient (m) line of (refer to Figure 3.25). The detail calculation is shown in Appendix C2.

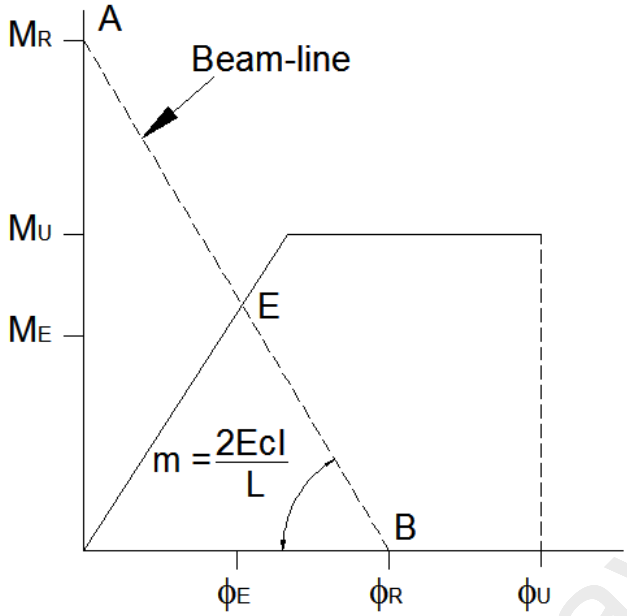


Figure 3.25: The gradient, m of beam line (Elliott *et al.*, 2003)

3.4.3 Connection Classification

For the connection classification based on Monforton's fixity factor (γ), the value of fixity factor (γ) is obtained from Equation (3). The detail calculation is also shown in Appendix C2.

CHAPTER 4: RESULTS AND DISCUSSIONS

4.1 Introduction

In this chapter, the results obtained from the experiment are presented in the form of tables and graphs. Three (3) similar specimens namely BIC 1, BIC 2 and BIC 3 were tested under reversed cyclic loading. The results are discussed in these nine (9) subtopics below. The subtopics are:

i. Material testing (Section 4.2)
ii. Moment rotation (M-ϕ) relationship (Section 4.3.1)
iii. Load displacement relationship (Section 4.3.2)
iv. Load strain curve (Section 4.3.3)
v. Connection Classification (Section 4.3.4)
vi. Failure modes and cracks pattern (Section 4.3.5)
vii. Analytical result (Section 4.4)
viii. Comparison of result between analytical and experimental method (Section 4.5)
ix. Discussion (Section 4.6)

4.2 Material Testing

4.2.1 Sika Grout 215

For grouting, Sika grout 215 with the strength 50 N/mm^2 at seven (7) days for flowable mixing and 65 N/mm^2 at seven (7) days for pourable mixing is used to fill the dowel holes in precast beam. For this connection, pourable mixing was selected. The compressive strength test were carried out using 3000 kN Concrete Compression Machine (Model : Controls) and with accordance to MS 26: Part 2: 1991 Section Three

(Method for Determination of Compressive Strength of Concrete Cubes). The cube test result at seven (7) days for grout is shown in Table 4.1 below.

Table 4.1: Grout strength for the specimens

Connection	Specified cube strength N/mm^2	Actual cube strength, N/mm^2 (At testing day)	
		Cube 1	Cube 2
BIC 1	65	63	69
BIC 2	65	62	67
BIC 3	65	64	66

4.2.2 Concrete

For concrete, it is designed for 40 N/mm^2 at seven (7) days. The concrete mix design is attached in Appendix B. Two types of test were carried out which are slump test and compressive strength test.

For slump test, MS 26: Part 1: 1991 Section Two (Method for Determination of Slump) was adopted. The slump test was carried out to ensure the workability of the mix. The slump test was passed where the workability is within the ranged of 60-180mm as per designed.

For compressive strength test, the test were carried out using 3000 kN Concrete Compression Machine (Model : Controls) and with accordance to MS 26: Part 2: 1991 Section Three (Method for Determination of Compressive Strength of Concrete Cubes). The results for cube test at testing day are shown in Table 4.2 below.

Table 4.2: Concrete strength infill for the specimens

Connection	Specified cube strength N/mm^2	Actual cube strength, N/mm^2 (At testing day)	
		Cube 1	Cube 2
BIC 1	40	47	45
BIC 2	40	40	49
BIC 3	40	53	46

4.2.3 Tension Reinforcement (T16)

Tensile test was done for hand placement of two (2) no. T16 mm rebar x 770 mm length that anchored to the column using threaded splice couplers. Tensile test was carried out in accordance with MS 146: 2006 Clause 16 (Mechanical Properties – Tensile) using 2000 kN Universal Testing Machine (Model : Shimadzu). The results for tensile test are shown in Table 4.3.

Table 4.3: Tensile test results for tension reinforcement

Reinforcement	Diameter (mm)	Length of bar (mm)	Yield load (kN)	Tensile load / ultimate load (kN)
Bar 1	16	800	103.93	118.62
Bar 2	16	800	104.24	119.08

4.3 Results from Experiment

The general results for all the testing are shown in Table 4.4.

Table 4.4: Summary of results obtained from experiments

Connection	Dia. of tension bar	Moment at first crack, M_{CR} (kNm)	[1]Moment capacity, M_{RC} (kNm)	[2]Ultimate Moment, M_U (kNm)	Ratio M_U/M_{RC}
BIC 1	2 x 16	30	77.94	105.45	1.35
BIC 2		29		97.77	1.25
BIC 3		29		80.48	1.03
Average		**29.33**	**77.94**	**94.57**	**1.21**

[1] The calculation of M_{RC} is based on the equilibrium of all forces present in the connection (it is calculated at the face of the column). The internal level arm, z is the resultant of the various horizontal forces such as threaded dowel and reinforcing tie bars (Figure 4.1). It is assumed that the forces can contribute to the moment capacity. This calculation is without partial safety factors.

[2] Ultimate moment capacity of connection obtained from testing

From the results shown in Table 4.4, the maximum moment of connections, M_U is greater than calculated moment resistance, M_{RC}. The calculation of M_{RC} is based on the equilibrium of all forces in connection and it is calculated at the face of the column. An assumption is made for M_{RC} which is all structural components present at the column face had achieved their full yield capacity. It is also assumed that compressive stress in the concrete infill is equal to 0.67 f_{cui} (compressive strength of grout). By using this assumption, the average value of M_U/M_{RC} is 1.21 with the range from 1.03 to 1.35. The ultimate moment, M_U of BIC 3 is significantly low compared to BIC 1 and BIC 2.

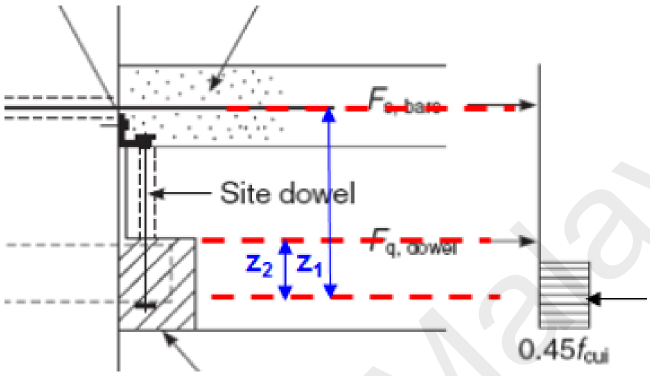


Figure 4.1 : Internal lever arm, z for reinforcement bar and dowel

4.3.1 Moment Rotation (M-ϕ) Relationship

The results for moment-rotation are interpreted in graph and it is shown in Figures 4.2 to 4.4. The graphs were plotted using data in Appendix D.

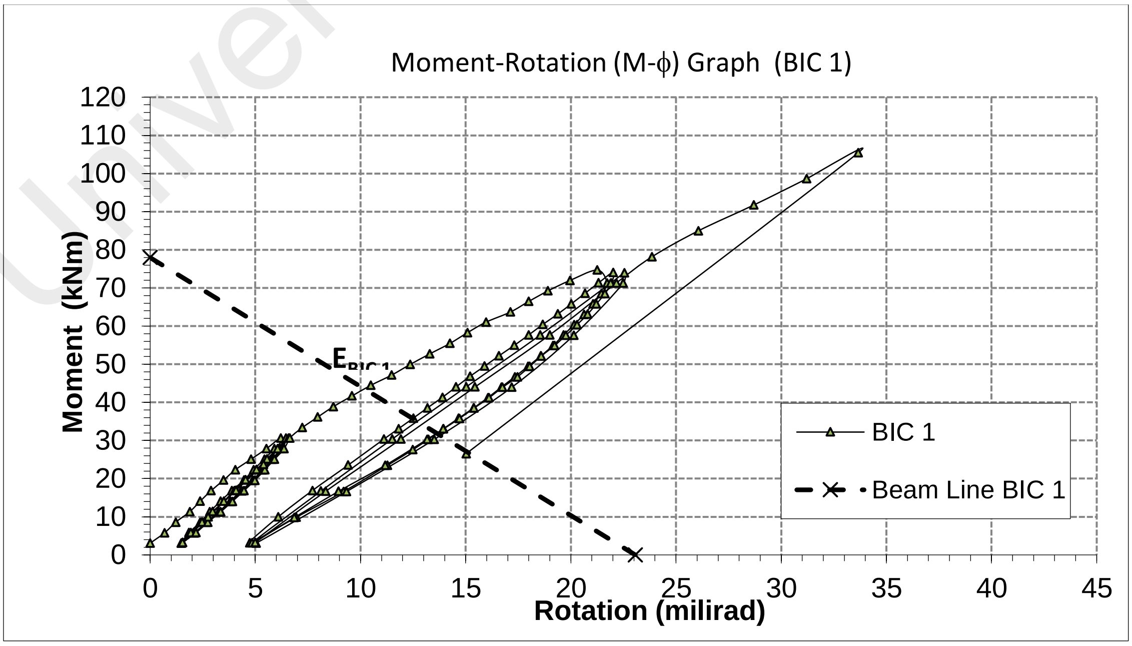


Figure 4.2: Moment-rotation (M-ϕ) graph for BIC 1

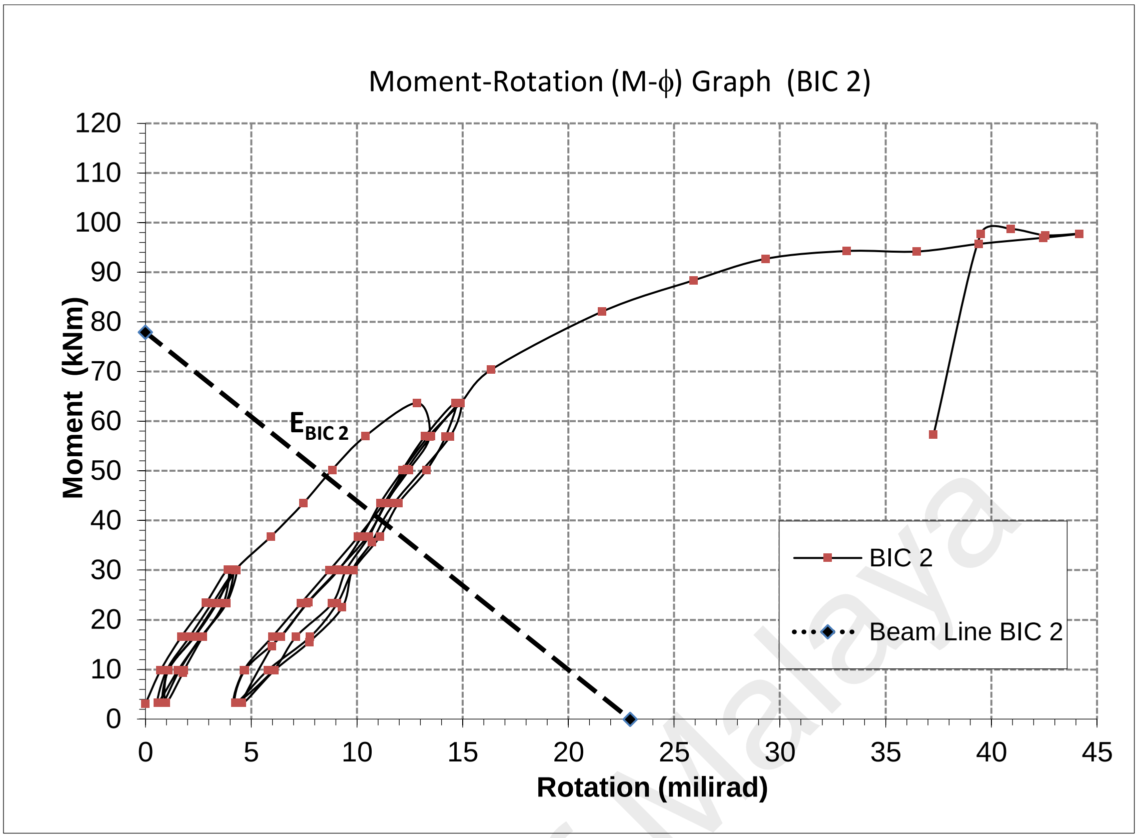


Figure 4.3: Moment-rotation (M-ϕ) graph for BIC 2

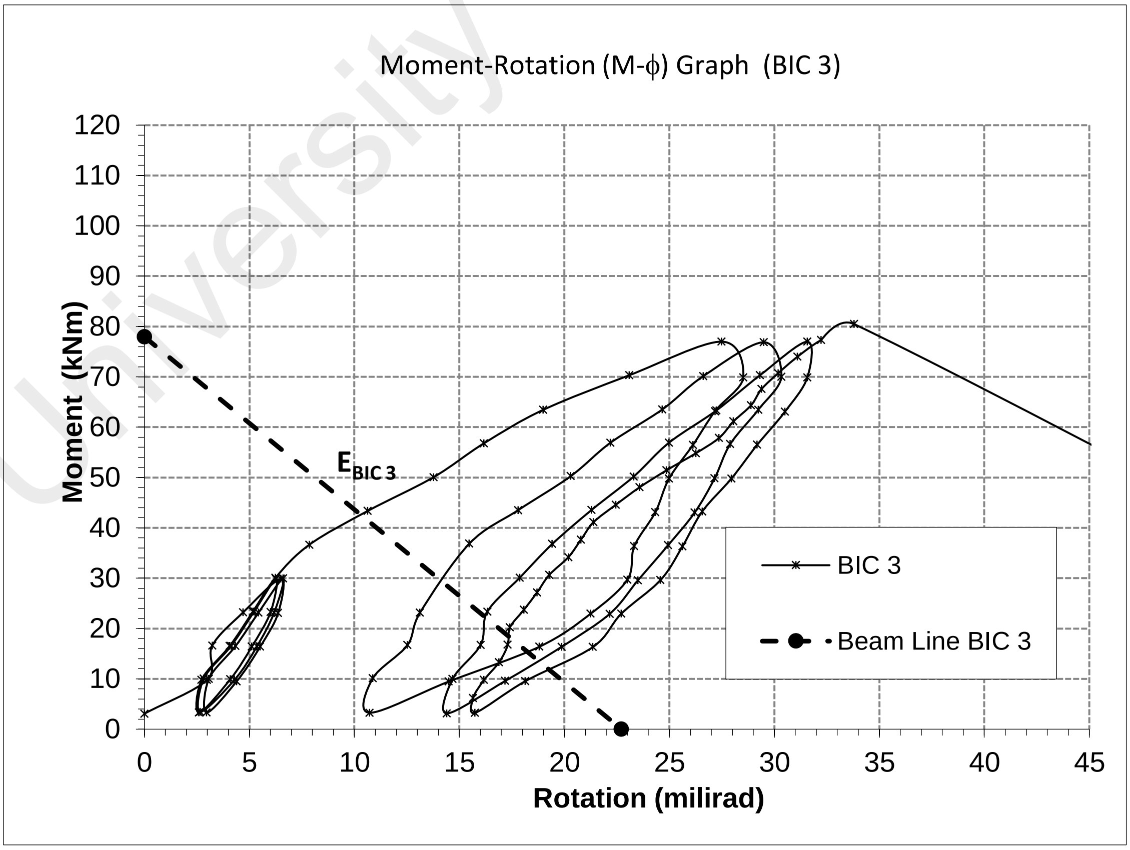


Figure 4.4: Moment-rotation (M-ϕ) graph for BIC 3

From the graph, the point of intersection (E) is determined where the end moment and the corresponding rotation of both beam and connection can be obtained. These intersections give the values of M_E and ϕ_E and it is summarized in Table 4.5. Then, this value is used in calculation of secant stiffness (S_E) from Equation (1) and stiffness factor (K_S) from equation (2) as given in Table 4.5.

Table 4.5: Results obtained from M-ϕ graph

Connection	[1]Moment at point E (M_E) (kNm)	Rotation at point E (ϕ_E) (milirad)	Secant stiffness (S_E)	Stiffness factor K_S
BIC 1	43.0	10.39	4.14	0.610
BIC 2	48.1	8.70	5.53	0.820
BIC 3	42.0	10.6	3.96	0.577
Average	**44.4**	**9.90**	**4.54**	**0.669**

[1] End moment and end rotation of both beam and connection / allowable moment capacity of a connection

The M-ϕ plots for BIC 1, BIC 2 and BIC 3 specimens are shown in Figure 4.5.

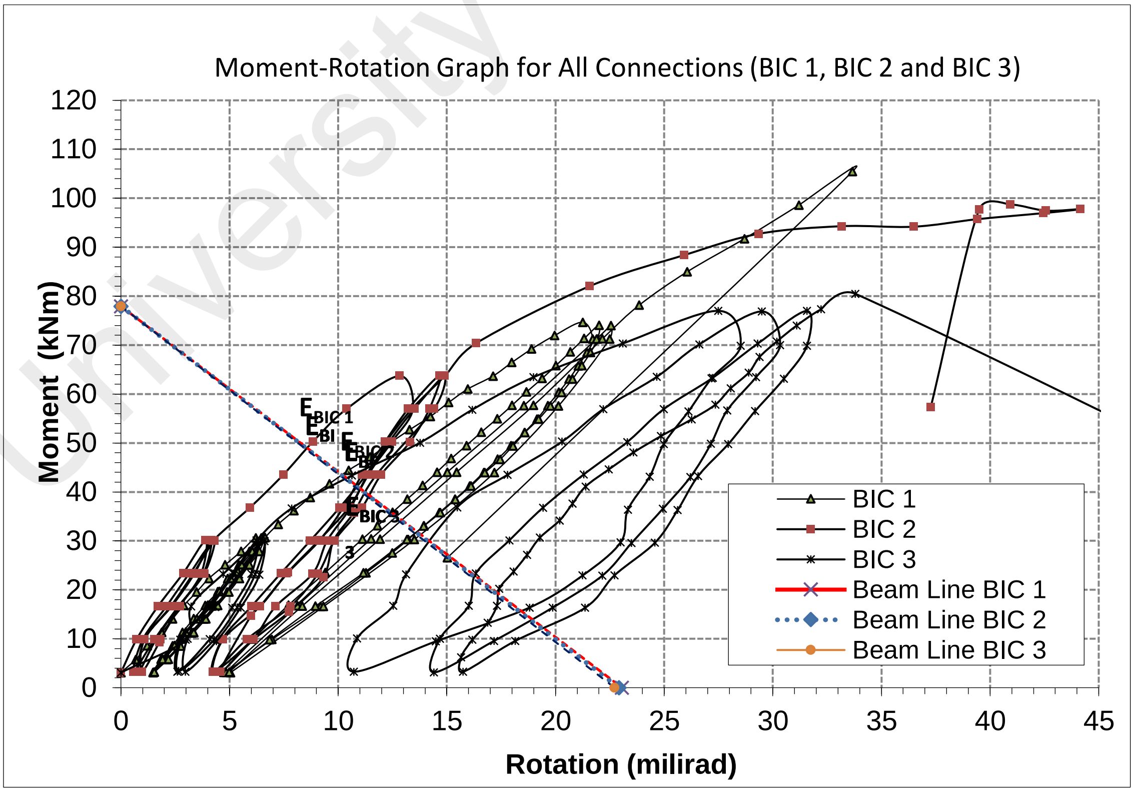


Figure 4.5: M-ϕ graph with beam-line for all specimens of connection

From the Figure 4.5 . All specimens failed beyond the beam-line which means that the connection has sufficient ductility and achieved required strength to be considered as a semi-rigid connection.

4.3.2 Load Displacement Relationship

Load displacement graph is drawn according to Park and Paulay (1975) (refer Figures 4.6 to 4.8. All the graphs were drawn based on displacement data obtained from LVDT 8. Besides, the ductility factors also calculated according to Park and Paulay (1975). The graphs were plotted using data in Appendix E.

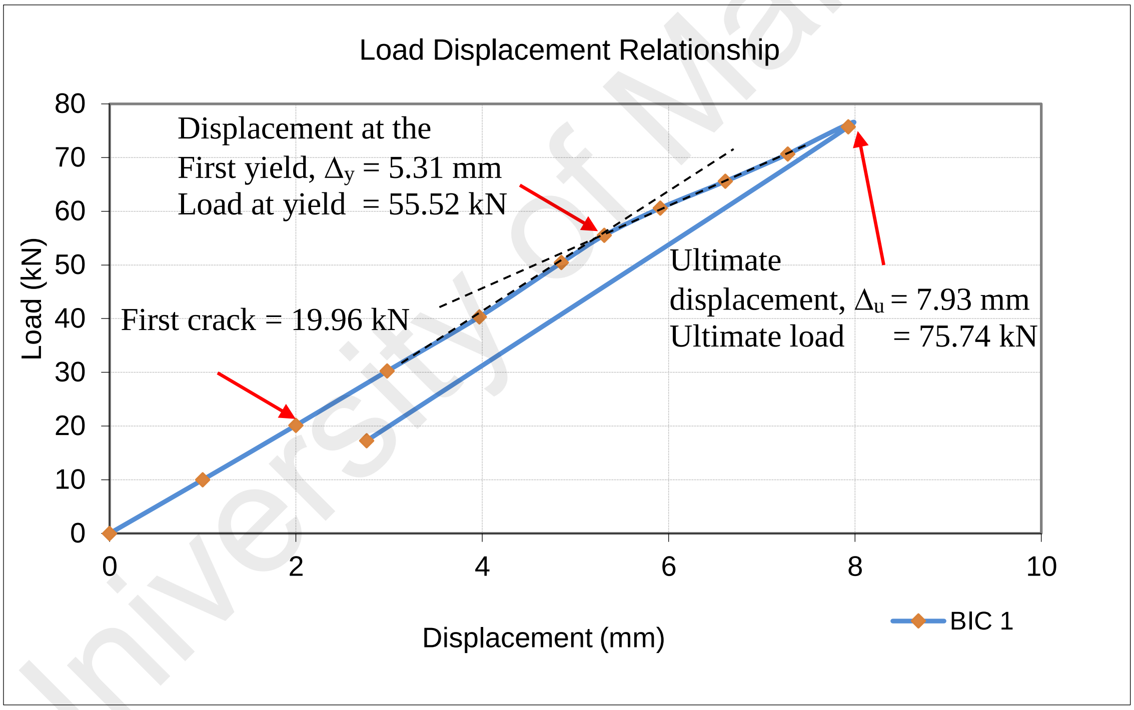


Figure 4.6: Load displacement graph for BIC 1

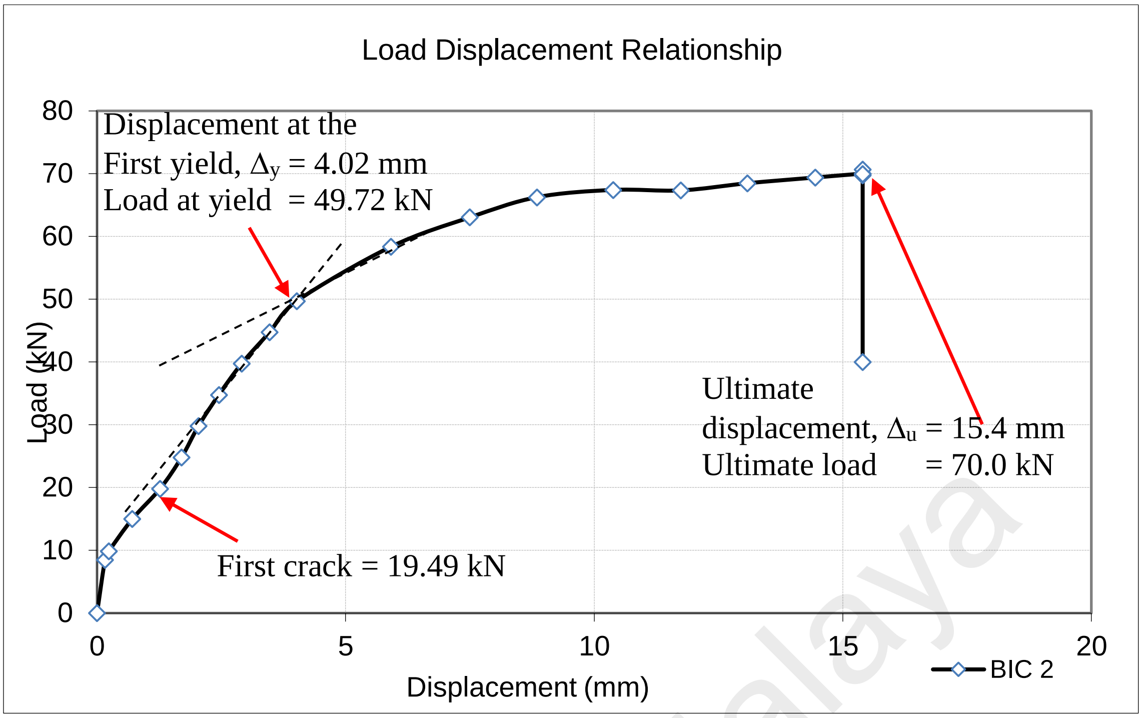


Figure 4.7: Load displacement graph for BIC 2

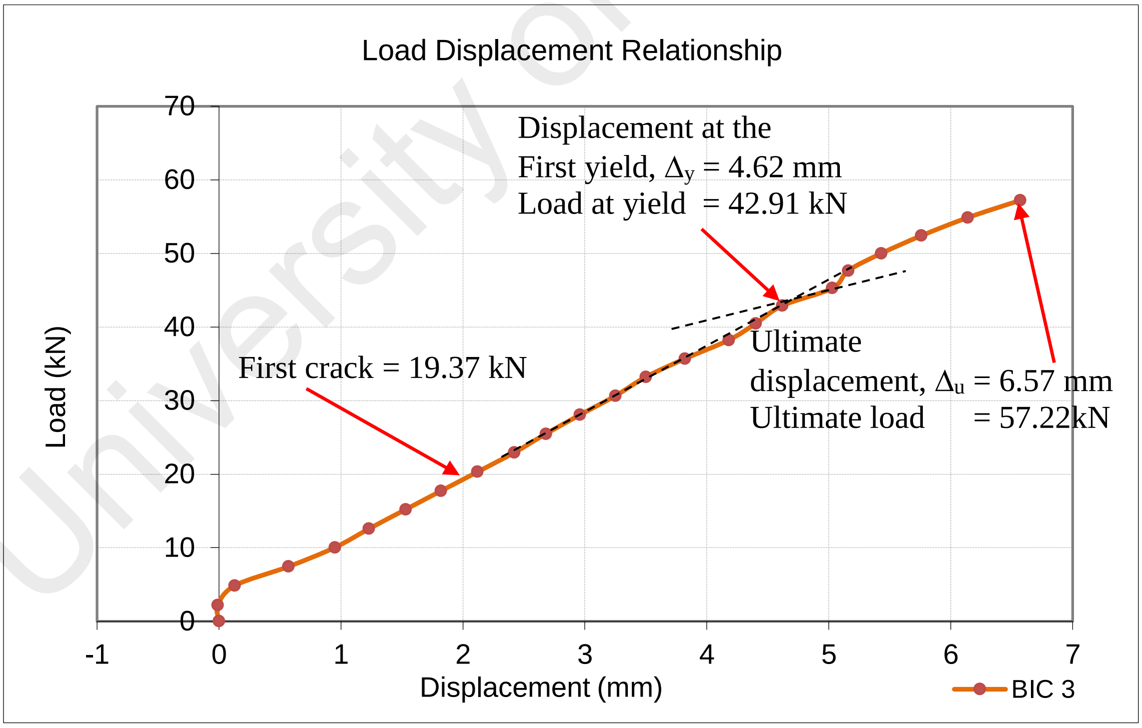


Figure 4.8: Load displacement graph for BIC 3

Based on the graph, the summary of the results are shown in Table 4.6.

Table 4.6: Summary of results from load displacement graph

Connection	Yield load (kN)	Ultimate load (kN)	Displacement at yield load Δ_y(mm)	Displacement at ultimate load Δ_u(mm)	Ductility factor $= \frac{\Delta_u}{\Delta_y}$
BIC 1	55.52	75.74	5.31	7.93	1.49
BIC 2	49.72	70.00	4.02	15.40	3.83
BIC 3	42.91	57.22	4.62	6.57	1.42

From the results, it is shown that BIC 1 and BIC 3 is at full elastic condition while BIC 2 achieve partial ductile (Refer Table 2.3). All the connections are capable to undergo inelastic deformation after the first crack. The connections also can maintain sufficient strength to support futher load and give warning of failure to prevent total collapse. The connections can be considered to have satisfactory ductility.

4.3.3 Load Strain Curve

The load strain curve is plotted based on load strain data obtained from steel strain gauge 5 (S5) and steel strain gauge 6 (S6) (refer Figure 3.21). The both strain gauges are fixed at two (2) points on left and right tension bar (T16). The curve plotted demonstrated the strain behaviour of tension bars during testing (refer Figure 4.9 to Figure 4.11). Based on the graph, the summary of the results is shown in Table 4.7. The graphs were plotted using data in Appendix F.

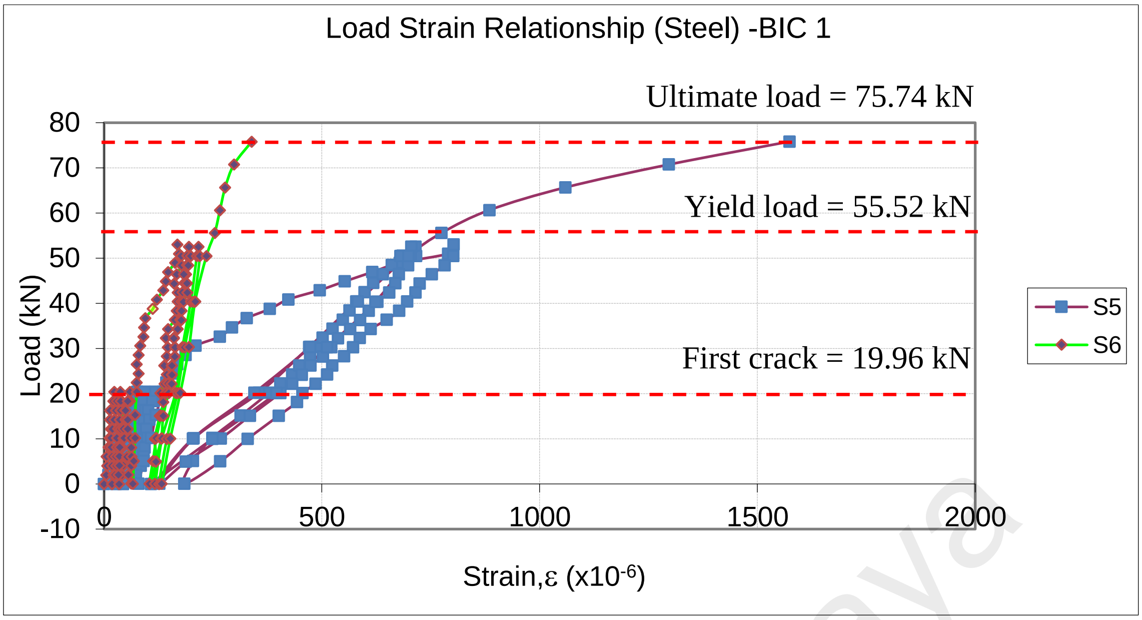


Figure 4.9: Load strain graph for BIC 1

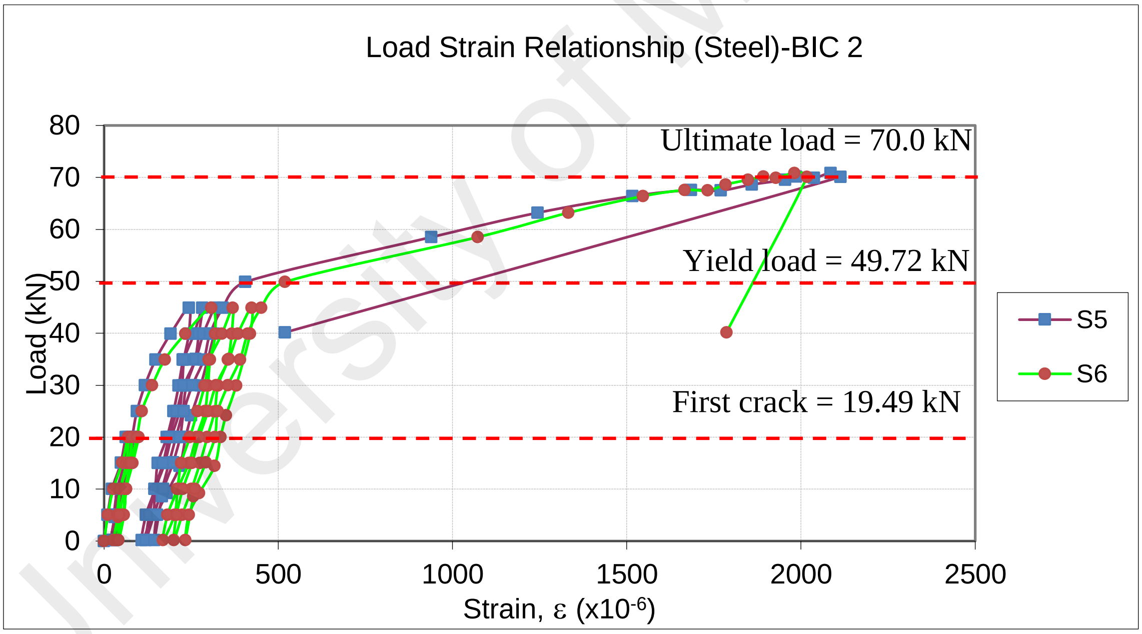


Figure 4.10: Load strain graph for BIC 2

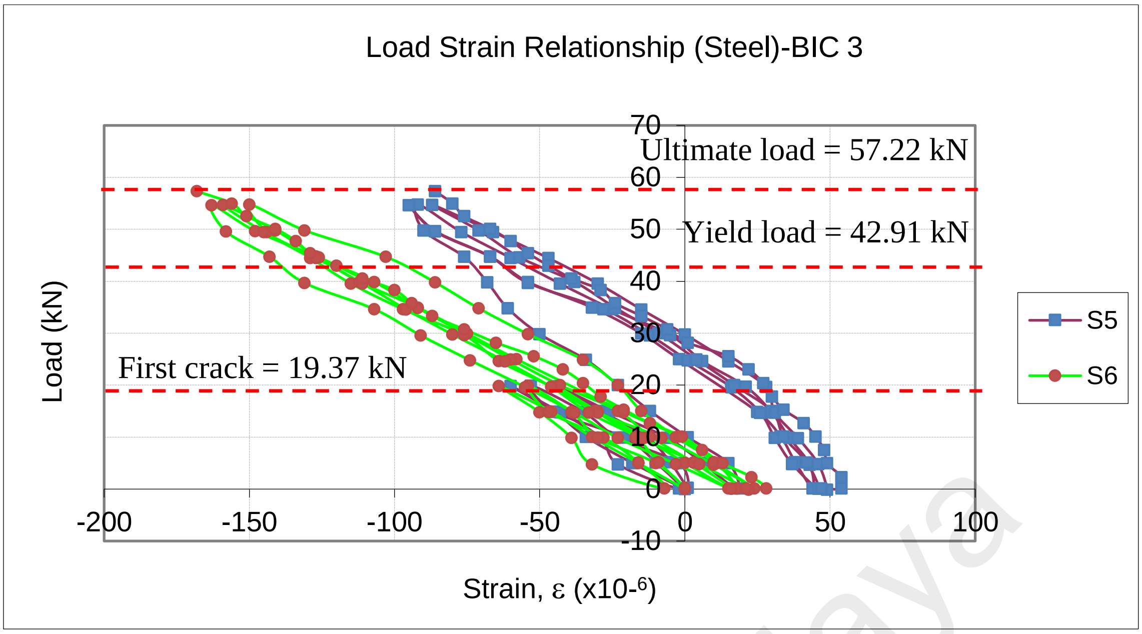


Figure 4.11: Load strain graph for BIC 3

Table 4.7: Summary of results from load strain graph

Connection	Strain Gauge	Strain at cracking load ε_{cr} (x10^{-6})	Strain at yield load ε_y (x10^{-6})	Strain at ultimate load ε_u (x10^{-6})
BIC 1	S5	345	775	1574
	S6	175	254	339
BIC 2	S5	218	405	2085
	S6	318	519	1981
BIC 3	S5	27	47	86
	S6	35	120	168

From the graphs, it can be seen that the load strain curve behaved in three (3) stages which are:

a) Before the first crack

The load strain curve of tension bars (T16) is in linear elastic

b) Between the first crack and initial yielding

The load strain curve of tension bars (T16) is tend to be nonlinear

c) After the initial yielding

The load strain curve of tension bars (T16) are approximately a horizontal straight line which means that the load remains almost the same while the strain still on increasing.

At the ultimate load (failure point) (refer Table 4.7), it is observed that the strain value for BIC 3 (ε_u : 0.000086 (S5) and 0.000168 (S6) are significantly low compared to BIC 1 (ε_u : 0.001574 (S5) and 0.000339 (S6) and BIC 2 (ε_u : 0.002085 (S5) and 0.001981 (S6). Bar slippage failure was happened in BIC 3 (discussed in 4.3.5). The bond slip in tension bars was happened at the early testing indicates by low increment of strain values.

In terms of stiffness, it is observed there are losses of stiffness for BIC connection. The changes in the slope of the curve (load strain) indicate there are losses of stiffness in the connection (indicates by black line for S5 and red line for S6). For BIC 1 and BIC 2, the losses can be obtained (refer Figure 4.12 and Figure 4.13) but for BIC 3, the losses can not be established since BIC 3 (refer Figure 4.14) having bond slip at the early testing. The purposed of loading and unloading condition load is to observed the stiffness of the connection

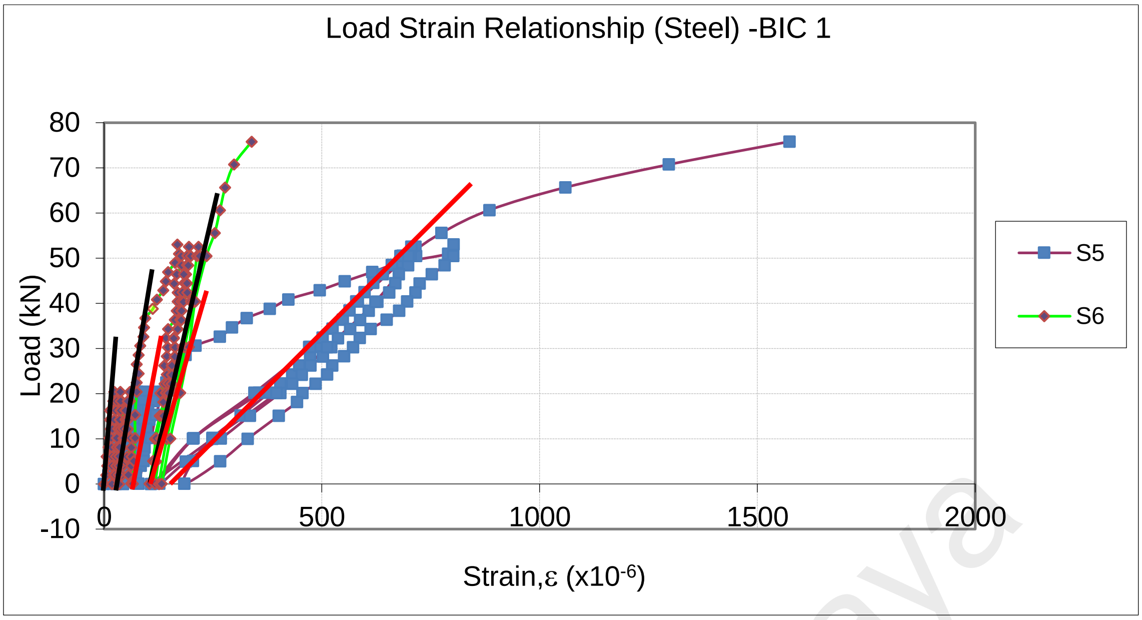


Figure 4.12: Stiffness losses of BIC 1

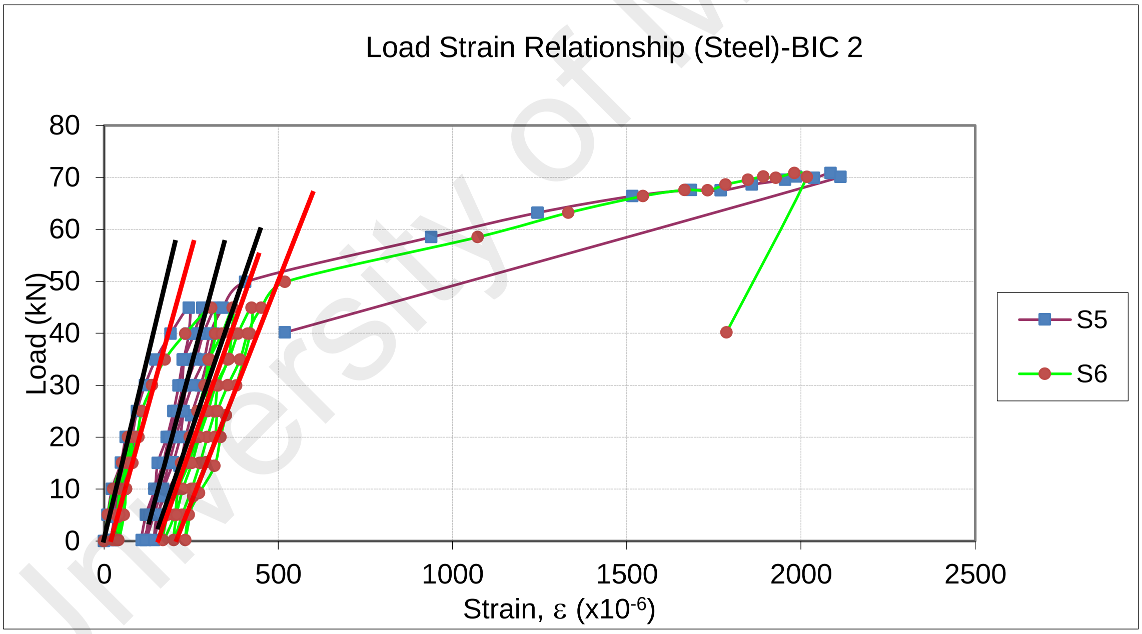


Figure 4.13: Stiffness losses of BIC 2

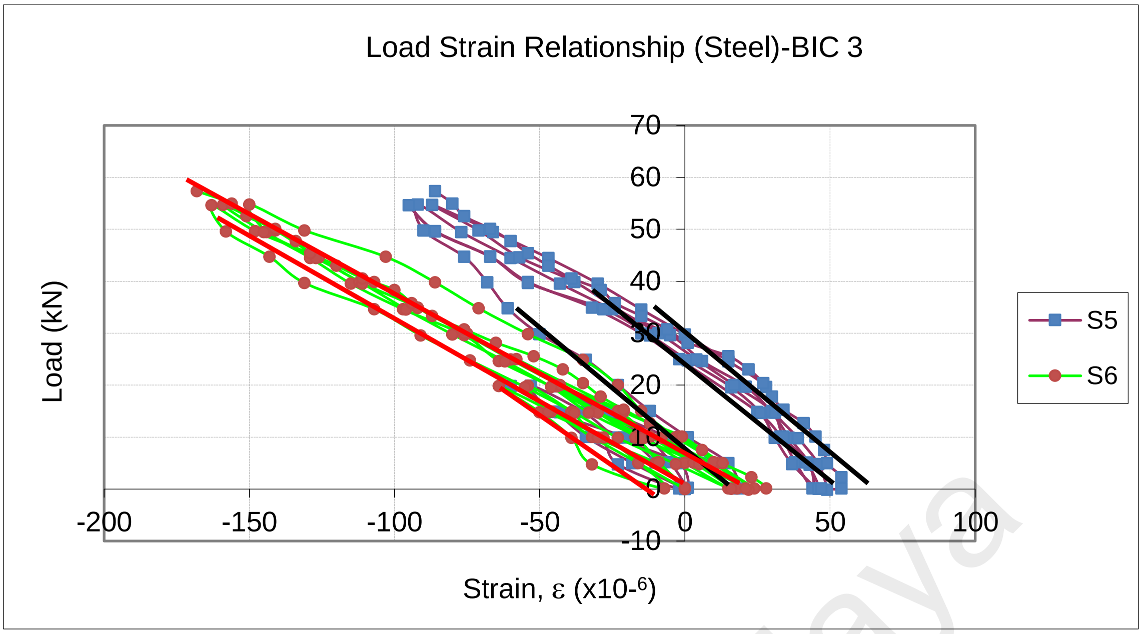


Figure 4.14: Stiffness losses of BIC 3

4.3.4 Connection Classification

According to Hasan *et al.* (2011), in order to quantify the rotational stiffness (S_E) of the connection, the Monforton's Fixity Factor (γ), as given in Equation (3) is adopted. The values from the calculation are shown in Table 4.8.

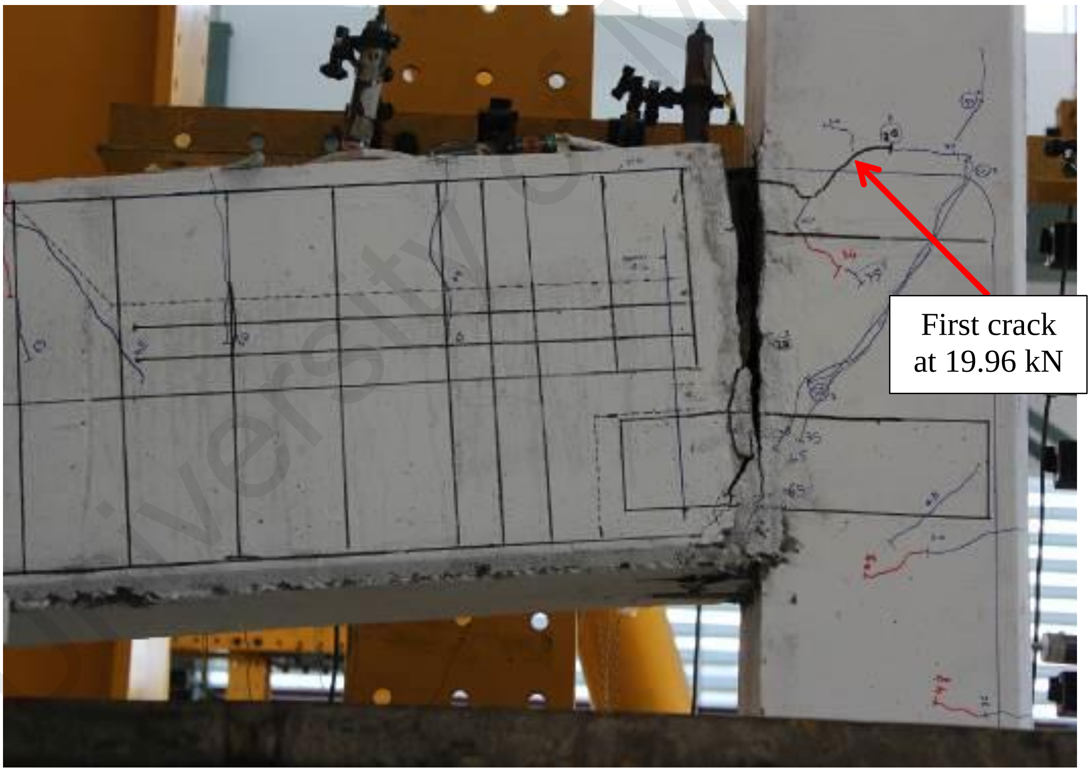
Table 4.8: Monforton's Fixity Factor value for BIC

Connection	Fixity Factor, γ
BIC 1	0.450
BIC 2	0.522
BIC 3	0.435
Average	**0.469**

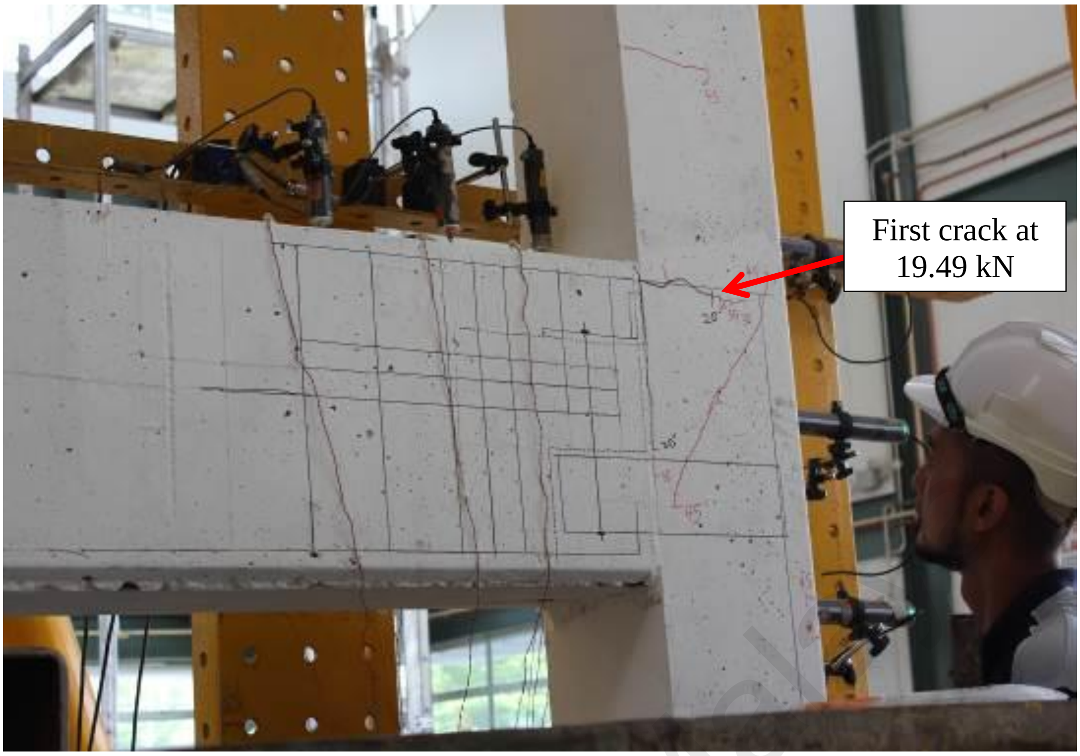
Then, this value is referred to the classification system for beam to column connection (Figure 2.24) which is reproduced after Ferreira *et al.* (2005). Based on the classification, the connection falls under Zone III which is classified as semi-rigid connection with medium strength.

4.3.5 Failure Modes and Crack Patterns

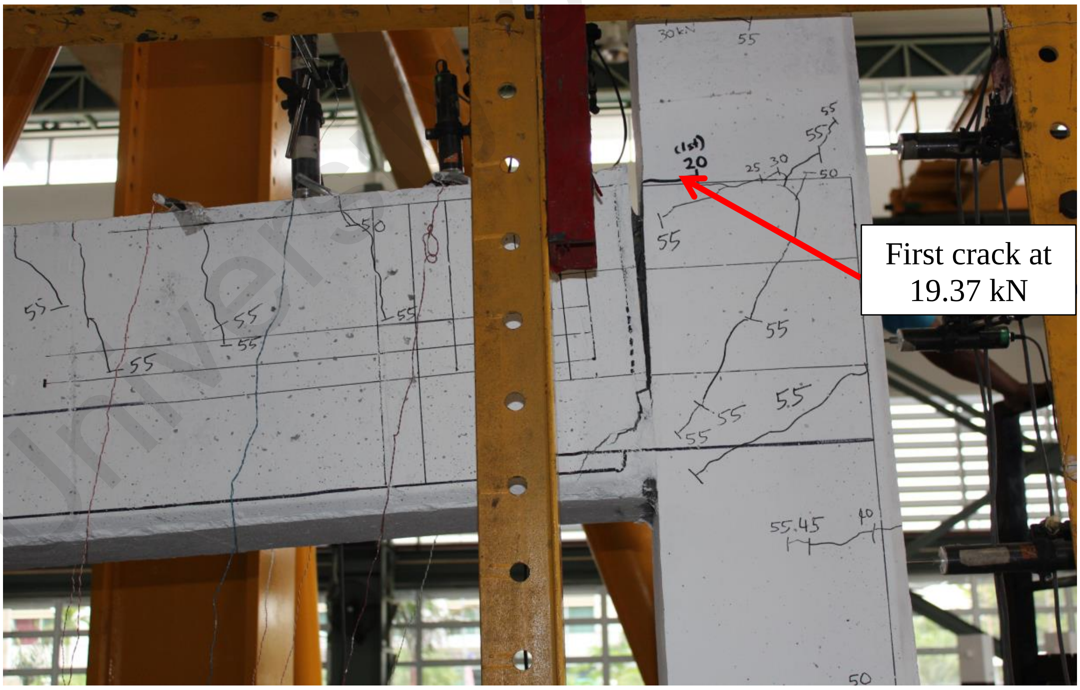
The crack patterns of BIC specimens are shown in Figure 4.15 to Figure 4.17. For all specimens, first cracks started to appear at 19.96kN (BIC 1), 19.49kN (BIC 2) and 19.37kN (BIC 3) respectively and all occured near the column. Possible failures that might occur at the connection region were highlighted by Meinheit and Jirsa (1981). For BIC, all specimens exhibited flexural cracking in the beam and column regions followed by diagonal cracking in the connection itself. Further load increments have extended the cracks. Based on the damage, plastic hinged had formed in the beam at face of the column. It means that ultimate moment resistance of the beam was reached. Besides, splitting cracks were also observed within the connection region.



(a) First crack at 19.96 kN for BIC 1



(b) First crack at 19.49 kN for BIC 2



(c) First crack at 19.37 kN for BIC 3

Figure 4.15: First crack for all connections happened at column

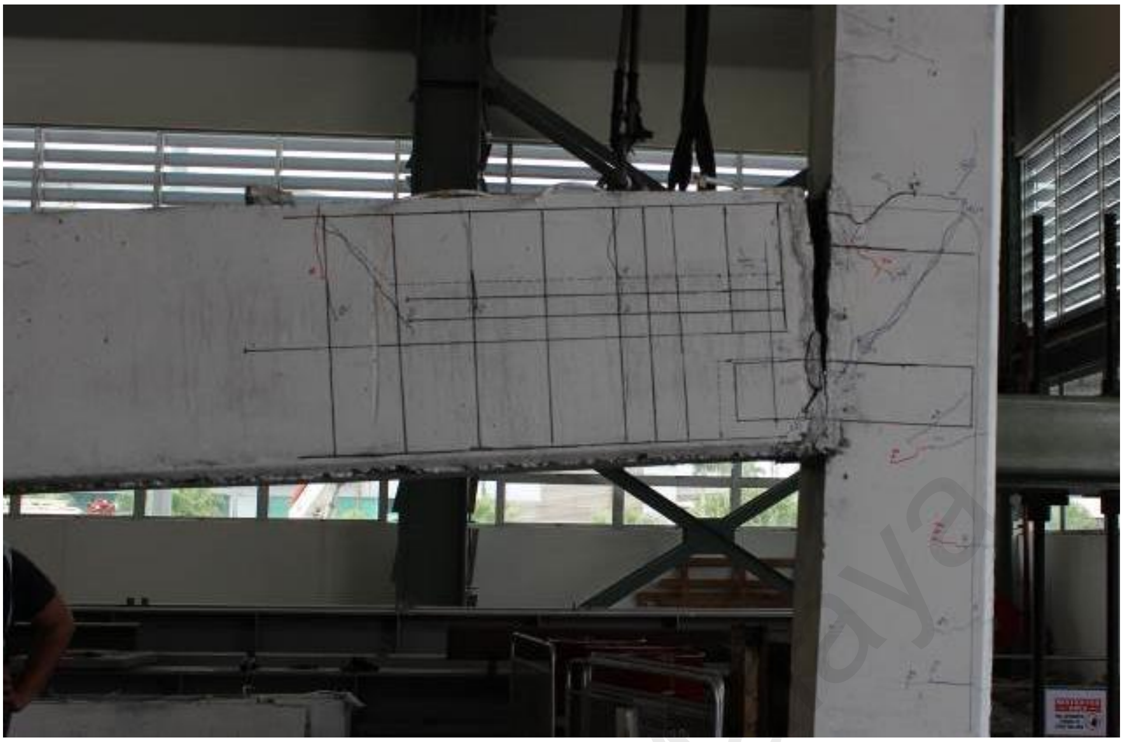


Figure 4.16: Damage specimens

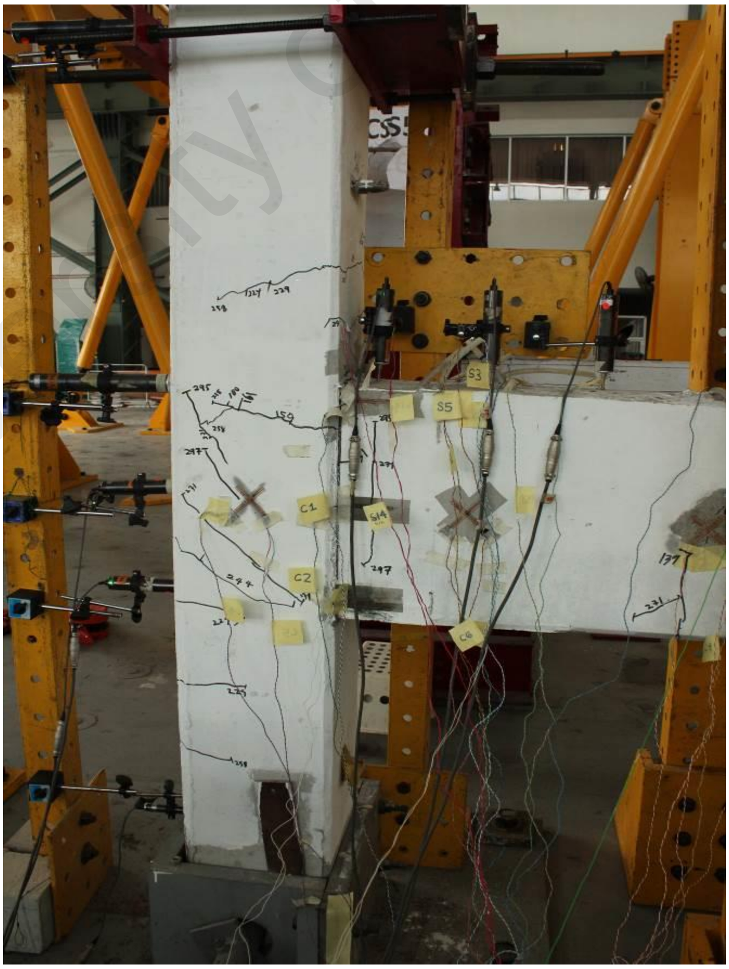


Figure 4.17: Cracks occurred at column region

The two additional tension bars (T16 rebar) that were used together with the splice connector had contributed significant to increase in moment resistance to this connection. Two modes of failure were observed, namely bar fractured and bar slippage. For BIC 1 and BIC 2, the failure mode was found to be bar fractured (see Figure 4.18) while for BIC 3 was bar slippage (see Figure 4.19 and Figure 4.20). The bars with fractured modes of failure (BIC 1 and BIC 2) can resist the moment up to 105.45 kNm while the bars with slippage modes of failure (BIC 3) can only resist untill 80.48kNm only. According to Shaedon (2012), steel bar is fractured when they achieved their ultimate capacity, while bar slippage is failed when the steel bar being pulled out from the splice connector. For bar fractured failure, the splice connector provides adequate interlocking mechanism to resist steel bar from slipping out. For BIC 3 bar slippage happened because of less efficient bond between steel bar and splice connector. The steel bars and splice connector was not well secured due to defect of fabrication work.

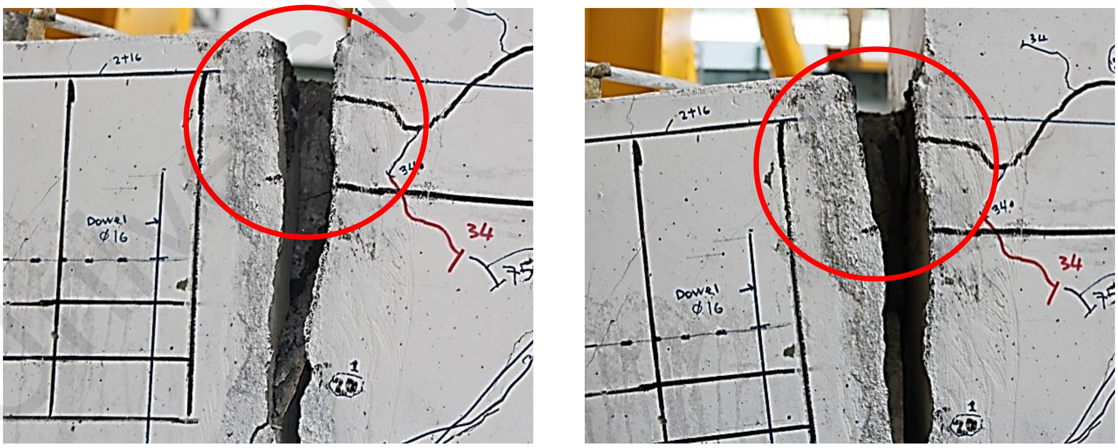


Figure 4.18: Bar fractured failure for BIC 1 and BIC 2

Figure 4.19: Bar slipped failure at the BIC 3 connection

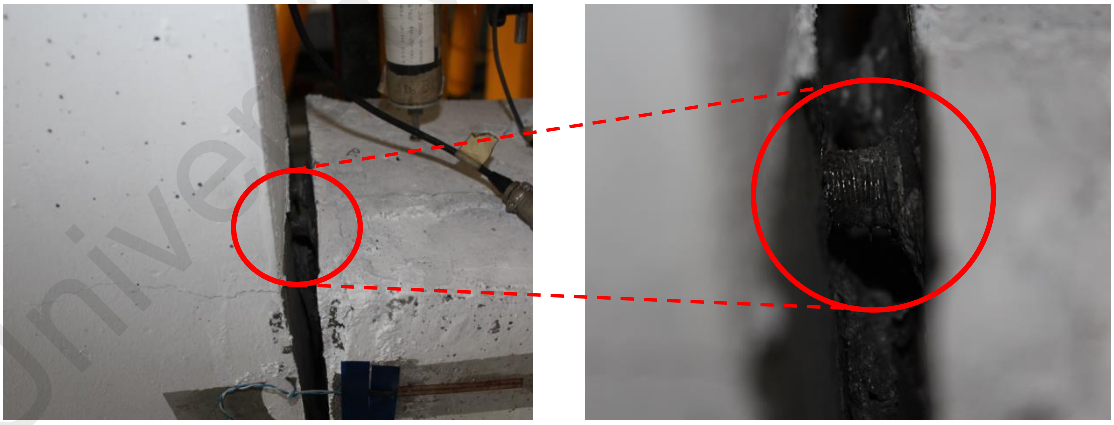


Figure 4.20: Bar slipped from the splice connector

4.4 Analytical Result

The results of analytical prediction of moment and rotation determine from Equation (5) to Equation (12) are shown in Table 4.9.

Table 4.9: Predicted moment resistance, rotation, stiffness of the connection, interception point and fixity factor for BIC connection

Connection	Moment capacity (kNm)	Connection rotation (ϕ) (milirad)	Secant stiffness (S_E)	Intersection at beam line kNm (E)	Fixity Factor (γ)	Classification (Figure 2.24)
Predicted	77.95	5.25	14.82	67.5	0.745	Zone IV

The predicted value shows that moment capacity of connection is 77.95 kNm and it falls in Zone IV which is semirigid connection with strength.

4.5 Comparison of the Result

The comparison between experimental result and analytical prediction is shown in Table 4.10 and also interpreted in graph (Figure 4.21)

Table 4.10: Comparison between experimental result and analytical prediction

Connection	Moment capacity (kNm)	Connection rotation (ϕ) (milirad)	Secant stiffness (S_E)	Intersection at beam line kNm (E)	Fixity Factor (γ)	Classification (Figure 2.24)
Predicted	77.95	5.25	14.82	67.5	0.745	Zone IV
Experiment	94.57	9.90	9.55	44.4	0.469	Zone III

The results show that the predicted value overestimates the experimental results (as shown in Figure 4.12). The predicted value may be increased if the contributions of mechanical parts (horizontal bolt, dowel and billet) are also included in the calculation of ϕ_c. In the current model, reinforcement bars is the dominant factor in the value for ϕ_c.

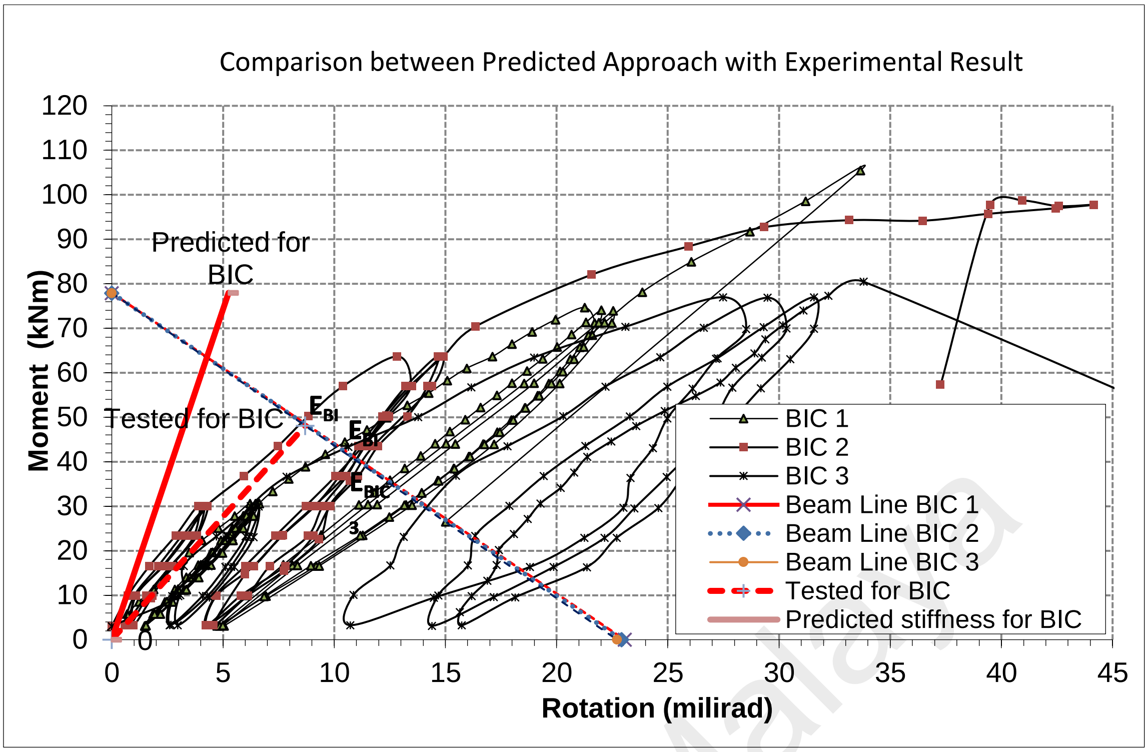


Figure 4.21 Experimental and predicted stiffness for BIC connection

4.6 Discussion

Among this three (3) test carried out, it is found that the reading for horizontal deflection cannot well established for BIC 1 whereas no relative deflection can be calculated. This is because the LVDT 7 toppled during the testing (refer Figure 4.22), while the reading for LVDT 6 is not well represented the beam deflection. So, only the reading captured from LVDT 8 is used to plot the graph. Thus, the plotted look smooth seems no failure occurred to this connection.

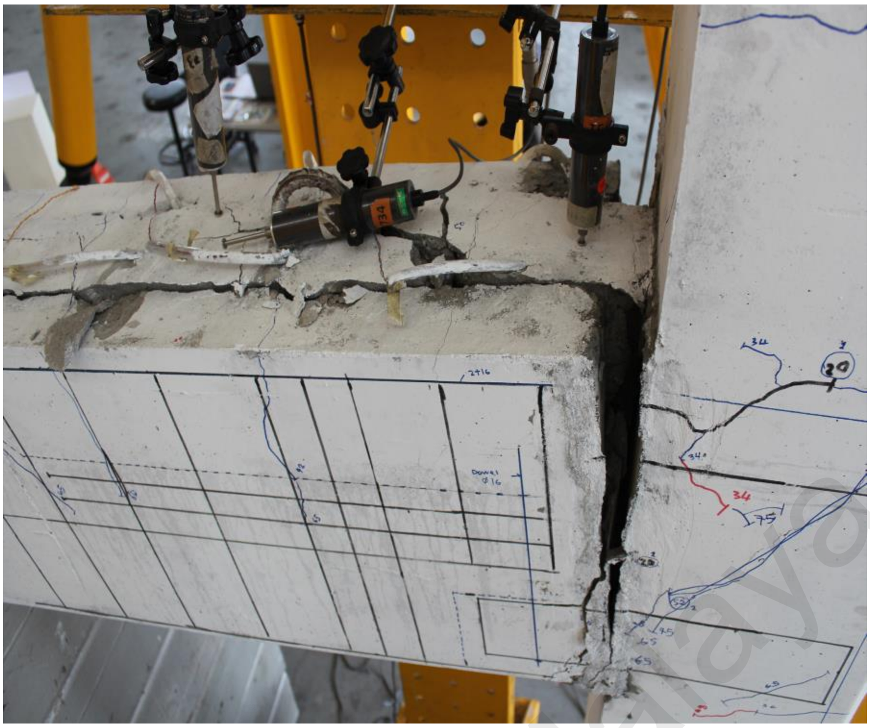


Figure 4.22: LVDT 7 toppled during testing

For BIC 3, the ultimate moment (M_U) obtained is the lowest compared to BIC 1 and BIC 2 and the value is quite significant. As mentioned earlier, this connection having bar slippage for the additional tension bar while the two others having bar fractured. It is due the two (2) no. of T16 (tension bar) is not well secure to the column splice effects from fabrication defect in beam. Thus, it is important to ensure this bar is well secured since this bar is a tension member that resists moment in connection.

Besides, for analytical method, it is needed to establish the contribution of mechanical parts in equation since this value also contributes to the important parameters in connection.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

Based on the experimental studies of a new proposed precast beam to column connection and analytical prediction, the following conclusion can be drawn:

i. The maximum moment resistance of proposed connections obtained from the test, M_U = 93.95 kN, and it is greater than the calculated moment resistance, M_{RC} (theoretical prediction) which is 77.94 kN, whereas the ratio connection failure load M_U/M_{RC} = 1.21.

ii. The analysis result shows that BIC is a semirigid connection with medium strength in Classification System based on Monforton's Fixity Factors,γ, (meaning γ = 0 pinned and γ = 1 fully rigid). The γ for the connection is 0.469 which falls in Zone III.

iii. The predicted value overestimates the experimental results. By neglecting the contribution of mechanical parts (horizontal bolt, dowel and billet) in the calculation of ϕ_c, has increased the analytical results. Reinforcement bars are the dominant value of ϕ_c.

iv. The ductility of the connections are considered as satisfactory since the value for for ductility factor is greater than 1 (ductility factor = 1 : full elastic condition). The results from testing obtained that ductility factor BIC 1 = 1.49 , BIC 2 = 3.83, BIC = 1.42.

v. In terms of connection behaviour, BIC connection has sufficient ductility and achieved required strength to be considered as a semi-rigid connection since it fails beyond the beam-line. Plastic hinge formation was observed in the beam; hence the ultimate moment resistance of the beam was reached. Besides, the two (2) no. of T16 rebar (tension bar) which were anchored to the column using thread splice coupler play an important role in determining the moment resistance of the connection. The bars with fractured modes of failure (BIC 1 and BIC 2) can resist the moment up to 105.5 kNm while the bars with slippage modes of failure (BIC 3) can resist till 79.45 kNm only.

5.2 Recommendation

The recommendations for future study for precast beam to column connection are:

i. All the instrumentation used for the testing should be properly setup to avoid errors during experimental.

ii. Further study to develop more theory of analytical as an alternative to predict the behaviour of connection. This will give some advantages/ help to designers who are unable to do experimental works.

iii. Further design and testing for different type of connection which can resist moment and also to fulfill architectural demand are recommended to be carried out.

iv. Parametric study on the behaviour of precast beam column connection using finite element method.

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