EXPERIMENTAL INVESTIGATIONS ON NOVEL DOUBLE-SLEEVE TUBEBOLT MOMENT CONNECTION TO CONCRETE FILLED TUBULAR COLUMN

MAHDI ZEINIZADEH JEDDI

FACULTY OF ENGINEERING UNIVERSITY OF MALAYA KUALA LUMPUR

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MAHDI ZEINIZADEH JEDDI

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Name of Candidate: Mahdi Zeinizadeh Jeddi

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EXPERIMENTAL INVESTIGATIONS ON NOVEL DOUBLE-SLEEVE TUBEBOLT MOMENT CONNECTION TO CONCRETE FILLED TUBULAR COLUMN

ABSTRACT

Concrete-filled tubular (CFT) columns have been commonly used as part of special moment-resisting frames in high-rise buildings for decades. Blind-bolts mechanical systems are widely adopted for on-site bolting of open beam to tubular column connections. The common types of blind bolts are Hollo-Bolt (HB) and its modified version, Extended Hollo Bolt (EHB). However, EHB bolts could not provide sufficient moment-resistance capacity and exhibited several shortcomings such as bolt slippage, low load transfer mechanism, bolt elongation and column face bending. A new blind bolt called Double sleeve-TubeBolt was proposed in this research to solve the above mentioned problems. The first part of this research was experimental pull-out tests of individual bolt under monotonic loading to obtain the performance of the TubeBolt and EHB bolt. The force-displacement, anchorage and elongation behavior of the bolts were determined. The test specimens were varied according to type of bolts, bolt diameter, concrete confinement and column wall thickness. For the second part of this study, six half scale cruciform specimens, connecting an open beam to CFT column with the novel blind bolt connection were tested under cyclic loading to investigate the seismic performance of the connections. The effects of different parameters on the behavior of the connection, such as bolt type, bolt grade and stiffeners at the top and bottom of the endplate were investigated. The failure modes, hysteretic performance, rotation capacity, strength and stiffness degradation, ductility and energy dissipation of the connections were analyzed. The pull-out test results indicate that the tensile capacity of the TubeBolt is 2.5 times higher than the existing EHB bolt. In addition, the Tubebolt has better

anchorage and the elongation has reduced to 68% compared to the existing EHB bolt. The cyclic test results of the cruciform specimens demonstrate that the new blind bolted connections exhibit a large hysteretic enclosed area, good ductility, and excellent energy dissipation compared to the EHB connection. The TubeBolt has achieved better performance compared to the EHB bolt, with 36% and 24% increase in the maximum strength and initial stiffness, respectively. No column face deformation and concrete damage is observed in the test. The results prove that the proposed TubeBolt connection satisfies the seismic provisions and ductility design requirements to be utilized in moment-resisting frames in seismic zones.

Keywords: TubeBolt connection, Concrete-filled tubular (CFT) column; Cyclic loading; Seismic performance; Ductility.

EXPERIMENTAL INVESTIGATIONS ON NOVEL DOUBLE-SLEEVE TUBEBOLT MOMENT CONNECTION TO CONCRETE FILLED TUBULAR COLUMN ABSTRAK

Tiang tiub terisi konkrit (CFT) lazimnya digunakan sebagai sebahagian daripada kerangka rintangan momen pada bangunan bertingkat selama beberapa dekad. Sistem mekanikal bolt buta digunakan secara meluas untuk menyambung rasuk terbuka kepada tiang tiub. Jenis-jenis bolt buta yang biasa digunakan adalah Hollo-Bolt (HB) dan versi yang diubah suai, Extended Hollo Bolt (EHB). Bagaimanapun, bolt EHB tidak dapat memberikan kapasiti rintangan momen yang mencukupi dan memperlihatkan beberapa kelemahan seperti gelinciran bolt, mekanisma pemindahan beban yang rendah, pemanjangan bolt dan lenturan muka tiang. Bolt buta baru yang dipanggil Double Sleeve-TubeBolt dicadangkan dalam penyelidikan ini untuk menyelesaikan masalah yang disebutkan di atas. Bahagian pertama dalam penyelidikan ini ialah ujian tarik-keluar bolt individu di bawah beban monotonik untuk mendapatkan prestasi TubeBolt dan bolt EHB. Kelakuan daya tarik - anjakan, tambatan dan gelinciran bolt ditentukan. Spesimen ujian dipelbagaikan mengikut jenis bolt, diameter bolt, pengasingan konkrit dan ketebalan dinding tiang. Untuk bahagian kedua kajian ini, enam spesimen bersilang separuh skala, menyambungkan rasuk terbuka kepada tiang CFT dengan sambungan bolt buta baru yang diuji di bawah beban kitaran untuk menyiasat prestasi seismik sambungan. Kesan parameter yang berbeza pada kelakuan sambungan, seperti jenis bolt, gred bolt dan pengukuh di bahagian atas dan bawah hujung plat diselidiki. Mod kegagalan, prestasi histeresis, kapasiti putaran, penurunan kekuatan dan kekakuan, kemuluran dan kehilangan tenaga sambungan telah dianalisa. Keputusan uji tarik-keluar menunjukkan bahawa kapasiti tegangan bagi Tubebolt adalah 2.5 kali lebih tinggi daripada bolt EHB. Selain itu, Tubebolt memberikan tambatan yang lebih baik dan pemanjangan Tubebolt

dapat dikurangkan 68% berbanding dengan bolt EHB sedia ada. Hasil ujian kitaran spesimen bersilang menunjukkan bahawa sambungan bolt buta baru memperlihatkan kawasan tertutup histeresis yang besar, kemuluran yang baik, dan pelesapan tenaga yang sangat baik berbanding dengan sambungan EHB. TubeBolt telah mencapai prestasi yang lebih baik berbanding bolt EHB, dengan peningkatan 36% dan 24% masing-masing dalam kekuatan maksimum dan kekakuan awal. Tiada ubah bentuk muka dan kerosakan konkrit yang diperhatikan di dalam ujian tersebut. Hasilnya membuktikan bahawa sambungan TubeBolt yang dicadangkan memenuhi syarat-syarat seismik dan keperluan reka bentuk kemuluran yang akan digunakan dalam kerangka rintangan momen dalam zon seismik.

Katakunci: Sambungan TubeBolt; Tiang tiub terisi konkrit (CFT); Beban kitaran; Prestasi seismik; Kemuluran.

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LIST OF SYMBOLS AND ABBREVIATIONS

- A_b : Bar area
- A_{brg} : Net bearing area of the steel bolt head
- A_n : Net area of end-plate
- A_s : Effective cross sectional area
- *b* : Width of the steel hollow tube
- b_{bf} : Width of beam flange
- C_{pr} : Factor to account for the peak connection strength
- *d* : Diameter of bolt
- d_{emb} : Effective bolt embedment
- d_h : Anchor bolt head diameter
- $(E_e)_i$: Elastic potential energy
- E_i : Energy dissipated by the structural members in the *i*-th loading cycle
- f_{cu} : Cube concrete compressive strength
- F_{nt} : Nominal tensile strength of bolt from the AISC Specification
- F_u : Minimum tensile strength of the yielding element
- f_{ub} : Ultimate stress of the steel bolt
- F_{up} : Specified minimum tensile strength of end-plate material
- F_{yp} : Minimum yield stress of the end-plate material
- F_y : Minimum yield stress of the yielding element
- f_c' : Cylinder compressive strength of the concrete
- *g* : Horizontal distance between bolts

 h_0

- Distance from the centerline of the beam compression flange to the :
 - centerline of the *i*th tension bolt row

h		Distance from centerline of compression flange to the tension-side outer
n_1	•	bolt row
k _{nc}	:	Modification factor
l _{dt}	:	Distance between <i>plastic</i> hinge locations
L_h	:	Distance between <i>plastic</i> hinge locations
M_{max}^i	:	Maximum moment at the i-th cycle of the story drift
M_n	:	Nominal moment strength of the CFT column
M_{pr}	:	Probable maximum moment at plastic hinge
M_u	:	Required flexural moment on the CFT column
M_y	:	Yield moment of the connection
N _{cb}	:	Concrete capacity
N_{pn}	:	Pull-out tensile strength
N _{sa}	:	Steel bolt strength
$(P_m)i$:	Average peak load of the relative cycle
$(P_{max})_i$:	Area under the load - displacement curve
$(P_{min})_i$:	Area under the load - displacement curve
R_s	:	Strength ratio
Ry	:	Ratio of the expected yield stress to the specified minimum yield stress
S _h		Distance from face of the column to plastic hinge
t	:	Tube wall thickness
t _{bf}	:	Thickness of beam flange
t_p	:	Thickness of end-plate
V_u	:	Shear force at the end of beam
Y_p	:	End-plate yield line mechanism parameter

- Z_e : Effective plastic section modulus of the section (or connection) at the location of the plastic hinge
- ϕ_c : Nominal axial strength reduction factor
- ϕ_n : Resistance factor for non-ductile limit states.
- $\psi_{c,p}$: Modification factor
- ψ_e : Modification factor
- μ : Ductility
- $\Delta \theta$: Failure rotation
- θ_y : Yield rotation
- ξ_{eq} : Equivalent viscous damping ratio

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University

CHAPTER 1: INTRODUCTION

The benefits of using CFT columns in tall buildings have been demonstrated. In the presence of compression load, the efficiency of the tubular column is better than the open column, where the load carrying capacity of CFT's can be increased remarkably. Figure 1.1 illustrates a CFT column bolted connection.



Figure 1.1: CFT column bolted connection (Jingfeng et al., 2013)

This study is present a novel bolt known as TubeBolt (Figure 1.2) for the moment connection between steel beam to the CFT column bolted connection to introduce the

connections with higher performance and more economical properties. Easy and fast prefabrication of the structures is the other aim of this study.

The TubeBolt connection can technically distribute higher load to CFT column and prevent from bolt elongation, bolt slippage and CFT column surface deformation. Thus, the moment-resisting capacity of the connection can be significantly enhanced due to the excellent anchorage performance of the TubeBolt.

A series of the pull-out test under monotonic loading are performed to investigate the behaviour of the novel TubeBolt. Moreover, seismic performance of the novel TubeBolt are examined through large-scale steel beam to CFT column moment connections.



Figure 1.2: Proposed novel TubeBolt

1.1 **Problem statement**

CFT columns exhibit excellent static and dynamic properties. Nowadays, the applications of CFT column are expanding in the constructions. The hollow sections are available in various shapes such as circular and square with different wall thickness. This type of column normally consists of high tensile strength steel with high compressive strength concrete. The lower steel consumption (Zhang et al., 1996), lower labour cost, easy and fast construction without using a mould for concrete casting could reduce the construction cost significantly. Moreover, because of the confinement of the concrete

inside the hollow column, the strength of the column is increased. In addition, the tube wall prevents from concrete spalling and results in higher maximum strength, stiffness and flexural capacity of the composite column.

In the past few decades researchers have conducted various steel beam to CFT column moment connections to achieve the required strength and stiffness for special moment resisting frames. Moreover, the proposed connections must be economical and easy for fabrication.

Researchers are more interested in semi-rigid connections because of ductility and energy dissipation capacity of these type of connections (Jingfeng et al., 2013). But this type of connection is still not being fully used due to the economical properties and moment-resisting connection efficiency. The bolt slippage, weak anchorage of the bolt inside the concrete core, elongation of the bolt and column deformation in the vicinity of the bolts hole were common problems for decades. These problems prevent the obtainment of maximum strength capacity of the bolt and reduce the moment-resisting capacity of the connection.

The proposed TubeBolt can achieve the maximum tensile strength and eliminate bolt slippage, column face deformation and reduce bolt elongation due to the excellent anchorage performance

Some codes such as ACI-318-08 (ACI 318-08, 2008), AISC 360-10 (ANSI/AISC 360-10, 2010), ASCE Task Committee (ASCE, 1994), BS5400 (British Standard, 1978; British Standard, 1984), Eurocode 4 (EN, 2004) and AJI (AIJ, 1997) were used to design of the CFT structures. The design guidelines for anchor bolts are also available in ACI 318-08 (ACI 318-08, 2008) and CEB (CEB, 1990; CEB, 1994; CEB, 1997). However, there is not an available code for steel beam to CFT column bolted connection. The lack

of the design codes can relate to the weakness of the CFT bolt connections, thin wall of the CFT column, impracticality, and higher cost due to the bolt slippage inside the concrete core.

To obtain an idead beam to CFT column bolted connection furthur studes must be performed. The higher fabrication cost, lower strength capacity of the bolt, lower moment-resisting capacity, bolt slilage, column face deformation, and lower ductility are most common problems.

1.2 Background

A concrete-filled tubular column has many advantages over steel open sections or reinforced concrete, such as delayed local buckling and high rotation capacity due to the effect of concrete confinement. Nevertheless, the wide utilizing of the beam to CFT column moment connection is related to adequate details being available for design and economical properties. Extensive studies have been conducted to propose suitable guidelines to design of steel beam to CFT column moment connections.

The main investigated parameters such as through beam to a CFT column connection, horizontal hunch connection, Rib plate stiffener, column-tree connection, vertical plate through the column, external T-shaped stiffeners, extending the web or flange through the CFT column and diaphragm connection, and stiffening plates around the CFT column, have been carried out by previous researchers to improve the performance and application of beam to CFT column connection.

Moreover, other studies have been conducted on the beam to CFT column load transfer mechanism including: extended end-plates bolted to the CFT column with steel rods passing through the column or beam passes through the joint with an extra bolted bracket, bidirectional bolted beam to column connections, blind bolted connection with end-plate flush end-plate or split tee, and internal diaphragm connection.

1.3 Objectives

The main objective of this study is to propose a novel blind bolt and examine its monotonic behaviour as well as examining the performance of beam to CFT column with novel blind bolt under seismic response. The specific objectives of this study are presented as follows:

- To propose a special novel blind bolt called TubeBolt as a connection fastener for CFT column.
- 2. To investigate the tensile monotonic pull-out performance of the TubeBolt embedded inside the concrete filled steel tube in terms of slippage, elongation, and column face deformation compared to existing EHB bolt.
- To investigate the seismic response of half-scale beam to CFT column with a new TubeBolt connection and examine the effect of the bolt grade and twin plates on the connection behaviour compared to existing EHB.

1.4 Scope of study

This study introduces novel blinded bolt known as TubeBolt to solve the CFT bolt connection usual problems. Moreover, a new type bolt connection is proposed to improve the design of the steel beam bolted moment connection to CFT column for moment resisting frames. A series of monotonic pull-out test with 30 numbers of specimens were conducted on the TubeBolt and compared with existing EHB bolt. The application of this bolt in beam to CFT column connection were examined through six half-scale test specimens under cyclic loading.

The main test parameters are as follows:

- For the pull-out test, specimens include bolt type, bolt grade, bolt diameter, bolt components and tube wall thickness. The specimens were experimentally tested under monotonic tensile loading to identify the anchorage performance of the bolts individually within the concrete filled tubes.
- For half-scale bolt connection to CFT column specimens includes bolt type, bolt grade, bolt components and twin plates to identify the seismic behaviour of the connection under lateral cyclic loading.

1.5 Outlines

The literature review of this thesis is organised in Chapter 2. The aim of the Chapter is to summarise the existing science to derive the gaps based on current research and prepare the objectives of this study. The review starts with the basics of the existing historical researches related to the steel beam to CFT welded connection performance. The chapter continues with the summary of current studies with regards to a steel beam to CFT welded connection. The principles of the fasteners mechanism inside concrete are examined to evaluate the blind bolted connections to CFT column. Due to the resemblance of the steel bars and bolts, the studies related to the steel bar reinforcement anchorage and bond stress performance are also reviewed. The main parameters that may influence the performance of the bolt are recognized and relative data is gathered to prepare an accurate experimental program to measure the TubeBolt components performance.

Chapter 3 represents the research methodology that is under investigation in order to evaluate the proposed TubeBolt performance individually and beam to CFT column bolt connection with respect to its various components. The experimental work is generally divided into three sections: manufacturing of novel Double-Sleeve TubeBolt and description of its components and advantages; monotonic tensile pull-out test to measure the load-slip relationships of individual TubeBolt components; moment-rotation relationships of new TubeBolt connections by means of cyclic lateral loading tests. The chapter involves experimental tests details, material properties of utilised steels, test setup, and instrumentations.

Description of experimental test results and arguments of the work is explained in Chapter 4. The tests results are presented in two sections; experimental pull-out tests under tensile monotonic loading and steel beam to CFT column moment connections under lateral cyclic loading. The performance of pull-out specimens is presented by means of load-slip relationship. Furthermore, the behaviour of moment connection specimens is also investigated in terms of hysteretic moment-rotation relationship. Moreover, the tensile behaviour of the pull-out specimens and seismic performance of the connections are discussed and compared to highlight the effect of the new proposed TubeBolt. The results will contribute to expanding the database of the moment-resisting beam to CFT column bolted connections.

Ultimately, Chapter 5 summarises the main findings and conclude the study.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

A review of different published literature is presented in this chapter. The literature studies are gathered in three sections. First, steel beam to CFT column welded connection are presented. The second section follows the existing information related to a steel beam to CFT column bolted connection. The third section discusses the current status of CFT column bolted connections in building construction industry. Furthermore, a review of existing pull-out tensile loading test and design guidelines for the bolt are provided. The purpose of this chapter is to highlight the gaps based on previous studies and define the subjects that need more research.

2.2 Steel beam to CFT column welded connection

2.2.1 Through-beam to CFT column connection

A beam through connection to CFT column under monotonic loading was introduced to improve connection behaviour and simplify the fabrication process (Azizinamini et al., 1993). In this method, steel I-beam passes through the hollow column to represent an interior joint in a structure then, beam flanges and beam web are weld to the column face. A verified finite element model was presented to examine the proposed design criteria of the connection. Another study summarised design guidelines for through beam to CFT column connection (Azizinamini et al., 1995). Moreover, seven 2/3 scale through beam connection specimens were designed and tested under monotonic loading to identify the failure mode and strength of the connections (Ahmed et al., 2001). The failure modes were described as column failure, beam failure, and joint shear failure.

The experimental results showed that tubular column, beam web and concrete had effect on the shear strength of the connection. In addition, through beam to CFT column

connection was a desirable rigid connection, compared with existing connection. Figure





Figure 2.1: Through-beam connection details

(Ahmed et al., 2001)

The experimental program that is conducted in this study prepared the basement of the design guideline to develop the design model, which is presented in other paper (Elremaily et al., 2001).

2.2.2 Beam to CFT column connection with the inner and outer diaphragm

Architectural Institute of Japan (AIJ, 1997) was presented design details for the new beam to column CFT connections. In this method, stiffener plates are welded to the inner and outer of the column and flanges. Different types of inner and outer diaphragm connection are depicted in Figure 2.2. However, manufacturing of inner diaphragm required specific instruments, complicated fabrication work, and higher cost.



Figure 2.2: Connection with the inner and outer diaphragm

Another program was conducted for steel beams to CFT column connections with floor slabs, including two interiors and two exteriors joints to assess the seismic performance of the connections (Cheng et al., 2007). In this study, the composite effect of the steel
beam and floor slab was evaluated. To prevent weld connections from the unexpected brittle failure, the seismic behaviour of new connection details such as the tapered flange or larger shear tab in the beam-end was performed as shown in Figure 2.3. Moreover, an analytical model for the force-deformation behaviour of the connection was developed and validated. The test results showed that the tapered beam flanges and lengthened shear tabs had moved the plastic hinges away from the column face. In addition, the shear tab has a lower effect on the shear transfer.



Figure 2.3: Connections with the interior and exterior connections

Further study was performed on two full-scale new internal-diaphragm connection to evaluate the seismic performance of the connection such as strength, stiffness, deformation, ductility, energy dissipation capacity and strain distribution (Ying et al., 2014c). Figure 2.4 shows the internal diaphragm connection. Experimental tests indicated that this connection could reduce the stress and strain condensation at the beamends, moved the plastic hinge away from the column face, and provide appropriate strength, stiffness, ductility, energy dissipation capacity and stable hysteresis behaviour in the seismic region.



Bottom flange

Figure 2.4: Connection with the internal diaphragm

Another research program was studied on four full-scale specimens of existing and proposed through-diaphragm connections to concrete filled square steel tubular columns under cyclic load to evaluate strength, stiffness, ductility and energy dissipation capacity (Ying et al., 2014a). The parameters in the study were the geometry of the through-



diaphragm, the arrangement of the weld access hole, horizontal stiffeners, and the connecting beam web to column method as shown in Figures 2.5.

Figure 2.5: Through-diaphragm connections detail

The results showed that moment-rotation hysteresis curves are constant and obvious strength deterioration and stiffness degradation was not found on the load-deformation hysteresis curves. Compared to the existing connection, the energy dissipation of the proposed through-diaphragm connections was improved and exhibit good seismic performance. Further study were presented two numerical models to forecast the flexural and shear strengths of through-diaphragm connections (Figure 2.6) between concrete-filled rectangular steel tubular (CFRST) columns and steel beams, respectively (Ying et al., 2014b). The first part of this study was to evaluate the moment transfer mechanism at the through diaphragm connections with yield line method. For the second part of this study, a numerical model for shear strength was presented based on the trilinear shear–deformation relationship. The contribution of frame mechanism in the panel zone was considered. Good agreement was found between numerical and experimental results for yield and ultimate shear strengths of the connections.



Figure 2.6: Shear strength of concrete core

2.2.3 Connections with different embedded elements

Six large-scale connections were tested under cyclic loading to investigate the behaviour of the different types of connections (Schneider et al., 1998) as illustrated in Figures 2.7 to 2.12. The results demonstrated that the welding of the connection-stub directly to the column face causes a large deformation on the column wall and results in fractures on beam flanges, column face and flange weld. The cyclic behaviour of the connection was improved by using external diaphragms to develop the forces of the flange to the column surrounding and distribute the bending strength of the beam. The embedded

elements have effectively transferred the beam flange force to the concrete core and reduce the stress concentration on the column wall. But, the type of embedded elements significantly effects on the behaviour of the connection. The bars deformed and welded to the beam flange and embedded into the concrete core. In this manner, the strength of the connection could be 1.5 times more than the plastic bending strength of the beam, and present stable hysteretic behaviour. Specimens with continuous flange plates without further members to anchor the flange plate into the concrete core exhibit lower force distribution. Passing the beam-stub through the CFT column was result in to transfer the full plastic bending strength of the beam to the column and represented ideal inelastic cyclic behaviour. Experimental results indicate that the type of connection detail significantly effect on the cyclic behaviour of the connection and the embedded elements move the plastic hinge away from column face.



Figure 2.7: Simple connection







Figure 2.9: Diaphragm plates connection







Figure 2.11: Continues flanges through column connection



Figure 2.12: Continues beam through column connection

2.2.4 Connections with reinforced bars

Ten large-scale experimental test under cyclic and monotonic loading were performed to investigate the behaviour of steel beam to CFT column connections (Beutel et al., 2001; Beutel et al., 2002). All specimens consisted of a direct connection of the beam to the column face using flange plates, and web cleat plates and reinforced bars welded to the top and bottom flanges. The bars then embedded into the concrete core. Figure 2.13 display details of the connection. Results indicated that the connection could move plastic hinge away from column face and exhibit ductile and stable behaviour throughout the test.



Figure 2.13: Embedded reinforce bars connection

2.2.5 Wide flange beam to CFT column connection with stiffening plate around the column

Seven experimental programmes were performed to evaluate the cyclic performance of the CFT connections with stiffening plate around column under cyclic loading (Park et al., 2005) as illustrated in Figures 2.14 and 2.15. Test results demonstrated that total rotation of 0.04 rad could be achieved which has satisfied the requirement for special moment resisting frames. This study had evaluated the force transfer mechanism at the connection using yield line method analysis.



Figure 2.14: connection with stiffening plate around the column



Figure 2.15: Beam to column connection with stiffening plate

2.2.6 T-stiffener connection

The cyclic behaviour of seven square CFT column to beam T-stiffener connections in terms of the ductility capacity were performed (Jae et al., 2008). All specimens were reinforced with T-stiffeners welded to the beam flanges as illustrated in Figure 2.16. The specimens obtained plastic rotations of 3% radian, which satisfies the requirements for special moment frame connections in the AISC seismic provisions.



Figure 2.16: T-stiffener connection

2.2.7 Steel reduced beam section to CFT column

In this study, hysteretic behaviour of five specimens of RBS (reduced beam section) and three specimens of weak-column connections without RBS steel beam to concrete-filled steel circular connections with an external ring were conducted (Wang et al., 2008).

The connection stiffening ring width, RBS arrangement and axial load level of the column were discussed. Figure 2.17 indicates the details of the connection. The results showed that obvious strength deterioration and stiffness degradation was not detected on the load-deformation hysteresis curves. Compared with weak-column connections, the energy dissipation of the RBS connections is improved and exhibit good seismic performance.



Figure 2.17: RBS Connection configuration

2.2.8 Connections reinforced with asymmetric lower diaphragms

A through-type beam to concrete-filled square column connection, reinforced with an asymmetric lower diaphragm was proposed to be used in weak-earthquake zones (Choi et al., 2010). A simple tension test was performed on the suggested lower diaphragms and the combined cross diaphragm in order to confirm their tensile behaviour.

Four full-scale specimens with the cross diaphragm, simple, horizontal T-bar, and vertical plate as the lower diaphragm were designed based on the ANSI/AISC SSPEC 2002 to investigate on the cyclic performance of the connections as shown in Figure 2.18. All specimens satisfied the 0.01 radian rotation capacity requirements for the AISC seismic provisions. The results indicate that lower diaphragms have adequate strength, stiffness, and deformation capacity.



Diaphragm in connection with different depth of beams at both sides.



Supposed connection detail.

Figure 2.18: connections with asymmetric lower diaphragms

2.2.9 Vertical plate passing through the column connection

A beam to build-up column connection with a new load path was presented to facilitate the fabrication process of the connection (Torabian et al., 2012). In this method, the internal horizontal continuity plate was eliminated to prepare sufficient space for concrete casting as illustrated in Figure 2.19. Two experimental tests were performed under cyclic loading to examine the cyclic response of the connection. The performance of the specimens was evaluated experimentally and numerically. The results showed that the connections had achieved more than a 0.06-rad story drift with sufficient strength degradation and satisfied recommendation for special moment resisting frames. In addition, the panel zone was remained approximately elastic throughout the test.



Figure 2.19: Vertical through plate connection details

2.3 Steel beam to CFT column bolted connection

2.3.1 Through bolted connection

A new beam to column bolted connections to CFT column was proposed (Wu et al., 2007; Wu et al., 2005) as shown in Figures 2.20 and 2.21. Six large-scale experimental tests under cyclic loading were performed. The column thickness and direction of the connections were varied. The beam end was reinforced with a flange wing plate and upstanding rib to prevent stress concentration on the beam end welded zone and increase the moment-resisting capacity of the connection.

A theoretical model was performed to determine equations of the ultimate shear strength, the stiffness and the yielding shear strength of the panel zone. The experimental results depicted that the bolted connections have sufficient strength, stiffness, energy dissipation and ductility to be utilised in seismic zones. Moreover, the story angular drift and the plastic angular displacement reach 7% and 5% respectively.



Figure 2.20: Through bolt connection with a flange wing plate and rib



Figure 2.21: Through bidirectional bolted beam connection detail

Another new connection to concrete filled circular steel column was proposed to improve the seismic behaviour of bolted joints (Li et al., 2009). The proposed bolted end-plate connections were designed and three full-scale experimental specimens connection were conducted to investigate the effect of the concrete floor slabs and reduced beam sections. Transverse ribs were welded to the column face to provide planes for end-plates as displayed in Figure 2.22. The high strength rods passed through the PVC tube located before concrete casting in the column and then tensioned to the demanded force while the concrete had reached to adequate compressive strength.

A nonlinear analysis and parametric study using the OPENSEES platform were proposed to simulate the seismic behaviour of the bolted end-plate connections. The experimental results show that all specimens have an inelastic rotational angle of more than 0.03 rad, which satisfies the AISC recommendation and exhibited good ductility and energy dissipation capability. In addition, the floor slab, which was connected to the steel beam by shear stud connectors, significantly effects on the behaviour of the connections.



Figure 2.22: Through bolted connection with transversal ribs end-plate

Further program was performed with four half-scale bolted connections to CFT columns under cyclic loading. For connection type 1 (Figures 2.23 and 2.24), the flat and curved extended end-plate were bolted to the CFT column where the rods were passed through the column. For the second type of connection, the beam was passed through the column and connected to it with bolted brackets without any welding between the beam and the column as shown in Figure 2.25. The experimental results indicated that all the connections displayed a desirable ductility and large rotation capacity without local buckling in the column face. Furthermore, the beam and rod passing through the column were significantly affected on the behaviour of the connections.



Figure 2.23: Through rod with End-plate connection



Figure 2.24: Rod through the square and circular column



(a) Square CFT column with through beam connection (Specimen No.3)



(b) Circular CFT column with through beam connection (Specimen No.4)

Figure 2.25: Through rod along with through beam connection to CFT column

2.3.2 Flush end-plate bolted connection to CFT column

A series of end-plate bolted moment connection to circular or square CFT columns were performed under monotonic and cyclic loading to investigate the static and seismic behaviour and failure modes of the connections (JingFeng et al., 2009a; JingFeng et al., 2009b). Figures 2.26, 2.27 and 2.28 illustrate the connections details.

Moment-rotation curves of the connections were generated to evaluate the stiffness, ductility, moment capacities and energy dissipation of the joints. The column section type and the thickness of the end-plate were the varied parameters.



Figure 2.26: Flush end-plate bolted connection to CFT column (cyclic)



Figure 2.27: Flush end-plate bolted connection to CFT column (Monotonic)

The results indicate that the proposed connections behaved in a semi-rigid and partial strength manner based on the Eurocode3 specification and showed rational strength and stiffness. Moreover, the rotation capacity of the connections exceed 70 mrad and satisfied the ductility requirements for seismic regions (JingFeng et al., 2009a).



Square column

Circular column

Figure 2.28: Inner view of flush end-plate bolted connection to CFT column

2.3.3 Top and bottom angle headed stud blind bolt

In this study four top and bottom angle connection to concrete filled square column were tested under monotonic loading (Agheshlui et al., 2011). The group performance of the headed stud blind bolts connections with different arrangement were studied and compared with the individual specimens. The headed stud blind bolt was the modified version of the Ajax bolt. The results indicate that the stiffness and strength of the connections enhanced. However, the blind bolts were subjected to the shear stress due to the significant deformation of the top and bottom angles. In addition, concrete cone failure was observed as illustrated in Figure 2.29.



Figure 2.29: Failure modes of the Top and bottom angle connection

2.3.4 Blind bolted connection with different column face stiffeners

In this study, six blind bolted connection to CFT column were tested under cyclic loading (Wang et al., 2016). To increase the moment-rotation capacity of the connections and reduce column face local buckling, four types of stiffener were proposed and performance of different stiffening methods were assessed. The stiffeners consisted of inner binding bars, welding two internal rings, externally welding two C-shaped channels, and internally embedding a short segment of I-section in the steel hollow column. Specimen details, arrangement and failure modes of the connections are shown in Figures 2.30 to 2.32.

The results show that the C-shaped channels and embedded I-section are effectively improved the performance of the connection, the joint strength and deformation capacity with large bending deformation which is developed in the end-plate. In addition, welded binding bars and internal rings stiffeners has a lower effect on the connection performance. However, the hysteretic moment-rotation curves of the specimens had depicted severe pinching (Figure 2.33).







Figure 2.31: Different types of the stiffener







Figure 2.33: Moment-Rotation hysteretic curves of the connections

2.3.5 Extended Ajax Blind bolted connection to CFT column

Another study was performed two series of experimental tests to investigate the tensile behaviour of headed stud blind bolts (Agheshlui et al., 2015; Agheshlui et al., 2011). The bolts were Ajax bolt version of EHB bolt as shown in Figure 2.34. Finite element modelling was also proposed to simulate the experimental results. The concrete cracking, crushing and development of cones and struts were also correctly predicted. Based on the experimental results it was concluded that anchored blind bolts can easily reach the ultimate tensile capacity of the equivalent high strength bolts if certain requirements are met. This means that anchored blind bolts can be designed for structural purposes using similar strength requirements to the ones of standard structural bolts. The bolts that were located at the sides of concrete filled tube reached their ultimate tensile capacity and failed by fracture. However, the bolts located in the middle of the tube, did not indicate a good anchorage performance and deformed the tube face.



Figure 2.34: Headed stud blind bolt arrangement

Seismic behaviour of extended end-plate blind bolted connections to circular or square CFT columns were investigated (Jingfeng et al., 2013). Four bolted moment-resisting connections under cyclic loading were tested. The parameter studies were the column section and end-plate type as shown in Figures 2.35 to 2.38. The failure modes, hysteretic performance, strength and stiffness degradation, rigidity classification, and energy dissipation of the connections were evaluated.



Figure 2.35: Extended end-plate connection details



Figure 2.36: Plan view of the extended end-plate connection

The reinforcing rebar is welded to the end of the shank to increase anchorage behaviour of the bolt within the concrete. The experimental results demonstrated that the blind bolted extended end-plate connections showed large hysteretic loops, good ductility, and excellent energy dissipation capacity. The rotation capacities were satisfied the ductility requirements for seismic regions.



Figure 2.37: extended end-plate connection deformation with the square column



Figure 2.38: Moment-rotation hysteretic curves of connections

2.3.6 The RMH (Reverse Mechanism HolloBolt) and Extended Hollo-Bolt (EHB) connection

Reverse Mechanism HolloBolt (RMH) and Extended Hollow Bolt (EHB) are modified version of Lindapter Hollo bolt (Lindapter, 2009) developed at the University of Nottingham to improve the stiffness of blind bolted connections to CFT column (Barnett et al., 1999; Barnett et al., 2000; Tizani et al., 2003) as illustrated in Figure 2.39.

For the construction of moment-resisting connections to CFT column an experimental investigation on EHB bolt component under monotonic pull-out was presented (Pitrakkos, 2012; Pitrakkos et al., 2013) as shown in Figure 2.40.



Figure 2.39: Different types of HB bolt



Figure 2.40: A view of the monotonic pull-out test setup

The aim of the study was to evaluate the component's level of pre-load and the nonlinear force-displacement relationship of the bolts. A total of 51 pull-out and 20 pre-load tests were conducted. The parameter studies were the shank (internal bolt) grade and length, the strength of the concrete core and the embedded depth of the bolts. The load transfer mechanism, the elongation of the shank, the slip of expanding sleeves and bolts anchorage capacity were investigated.

The test results indicated that the EHB bolt achieved the full tensile capacity by using 20 mm rigid plate. The compressive strength of the concrete core effected on the tensile stiffness of the EHB bolt component.

Another research was performed on six full-scale EHB bolt connection to concretefilled square column under cyclic loading and eight specimens under monotonic loading to investigate on stiffness of the connection, moment-resisting capacity and column face deformation (column face flexibility) of the EHB connection (Tizani et al., 2013a; Tizani et al., 2013b). Figure 2.41 illustrates the EHB bolt view inside the hollow column. The end-plate type, bolt grade (for cyclic test), loading history (for cyclic test), column thickness and concrete strength were varied in this experimental program.

Despite the authors claim about sufficient rigidity of the connection for seismic zones, the major local buckling on the column face and severe pinching on the hysteretic curves were observed as shown in Figures 2.42 and 2.43. This type of column is still not being fully used due to the economical properties and moment-resisting connections efficiency.



Figure 2.41: EHB bolt connection inner view



Figure 2.42: Failure mode of EHB bolt connection



Figure 2.43: Hysteretic moment-rotation curves of the connections

2.4 The status of steel beam to CFT column bolted connection in the building construction

The benefits through the use of CFT columns have been demonstrated especially in tall buildings according to many studies (Bergmann et al., 1995). This is because CFT connections have displayed excellent static and dynamic properties such as larger energy dissipation, better ductility, high stiffness, and strength. Not only that, the easy and fast connection, better fire resistance and the aesthetic views of tubular columns have made these CFT columns more appealing to designers and architects (Tizani et al., 2003). However, welded connection is still the common method to connect a beam to a CFT column. More specifically, various studies have shown that the welded connection used on these CFT columns are normally complicated in such a manner that extra-embedded steel bars and stiffeners are required to achieve sufficient moment capacity.

Since 1994 Northridge and 1995 Kobe earthquakes, many studies have revealed the catastrophic failures and brittle fractures on this type of connections. As a result of this, researchers are more focused on semi-rigid connections because of ductility and energy dissipation capacity of these types of connections.

Due to this, blind-bolt mechanical systems are widely adopted for on-site bolting of the open beam to tubular column connections (Barnett et al., 1999; Barnett et al., 2000; Klippel, 1998). The most common types of these blind bolts include, for example, Hollobolts (Lindapter, 2009) and extended Hollo-bolts (Pitrakkos et al., 2013). These types of bolts allow insertion and tightening from one end of the tube. The extended Hollo-bolt, otherwise known as EHB, on the other hand, is a modified version of the Hollo-bolt (HB) such that it could achieve more moment capacity in comparison with EHB. Typically, their dynamic and static properties like energy dissipation, ductility, stiffness, and strength are not satisfactory enough. Because of these reasons too, these blind bolts do not achieve the sufficient moment capacity as desired. Several shortcomings such as bolt slippage, bolt elongation, and column surface flexibility (deformation) in the vicinity of the beam-to-column connections, therefore, persist in these blind bolts.

The Hollo-bolts (HB) and the extended Hollo-bolts (EHB), generally do not have the sufficient ability particularly in terms of its anchorage function to connect the beam to the CFT column. Hence, it is more difficult for these blind bolts to transfer tensile load into the concrete core in their beam to CFT column connection. In the same vein, these blind bolts simply do not have the sufficient structure (for example, only one clamping member which is close to the column surface and small nut at the bolt end), particularly along their entire lengthwise extents thereof when it comes to load transfer.

Furthermore, both Hollo-bolts and extended Hollo-bolts, their structures tend to be more linear and continuous i.e. there is usually no segregation in their structures. Thus, these blind bolts are primarily prone to bolt elongation, among others (bolt slippage, column surface flexibility, to name a few).

Notwithstanding, whilst there is a nut at the end of the extended Hollo-bolt, it is still not sufficient in terms of anchoring the blind bolt for a beam to CFT column connection. In other blind bolts such as the Hollo-bolt, such an anchoring function is pretty much not existent in their structure.

Overall, there is a serious lack of anchorage mechanism and load transfer functions in the structures of these blind bolts. Furthermore, energy dissipating and load transfer functions should be uniformly or variably distributed or spread along the lengthwise extent of such a blind bolt.

Apart from the absence of such an anchoring function in most of the blind bolts, for the extended Hollo-bolt, the end anchor member is too small for transfer load. As such, the load carrying capacities of these blind bolts are equally insufficient particularly when it comes to these beam-to-column connections.

In light of the above, there appears a need for a fastener that connects structural members of a building that is sufficiently capable of overcoming all of the above drawbacks and shortcomings.

2.5 Embedded bar and bolt behaviour

Due to the strength of the bond between the concrete and the steel bar, the reinforced concrete is utilised in the structural design. The bond resistance helps to transfer tensile load from the embedded steel bar into the concrete core, steel bar anchorage, the stiffness of the structural member and crack control. Figure 2.44 shows the bond stress concept.

Many experimental tests were conducted to examine the bond between concrete and steel bar (Abrams, 1913; Eligehausen et al., 1982). All test results have demonstrated a relationship between bond stress and relative slippage of steel bar.



Figure 2.44: The concept of bond stress

The bond stress-slip relationship is a fundamental concept to predict the complicated interaction between concrete and steel bar. The bond-slip relationship for monotonic loading is presented in the CEB-FIP (CEB, 1990) and ACI 408-03 (ACI 408 Committee, 2003). The concrete-bar bond stress defines as a relative displacement function and illustrates as bond stress-slip relationship curve as shown in Figure 2.45.



Figure 2.45: Bond stress-slip relationship

RILEM (RILEM TC, 1983) recommended a test setup named pull-out test that is generally utilized to evaluate bond features as illustrated in Figure 2.46.



Figure 2.46: Schematic test setup arrangement for pull-out

Many researchers have conducted a pull-out test under monotonic loading and investigated on the bond mechanism and effective parameters on the relative stress-strain relationship. Some of the published works are presented below.

2.5.1 Concrete grade

The bond resistance of a reinforcing steel bar is known to be related to the compressive strength of concrete. Codes state that design bond stress is proportional to the square root of the compressive strength (BSI, 1997). Experimental studies have shown that bond stress between concrete and steel bars are modified by higher concrete compressive strength (Dancygier et al., 2010; Kankam, 2003; Mo et al., 1996). The concrete with higher compressive strength illustrated a sharp stress reduction after achieving maximum stress in the bond stress-slip curve. The concrete with higher compressive strength shows higher bond stress. Moreover, the failure mode of higher strength concrete is brittle compared with lower strength concrete (Ahmed et al., 2007; Alavi-Fard et al., 2004).

2.5.2 Embedment depth

A series of experimental tests was conducted to compare bond stress development between the ribbed (deformed) bar and concrete with normal and high compressive
strength (Ahmed et al., 2007). The study depicts that higher embedded depth of deformed bar results in lower slippage compares with concrete with normal compressive strength.

2.5.3 Rebar geometry

The deformed bar and plain bar are generally utilised to reinforce the concrete members. The deformed bar geometry is similar to the threaded shank and has rib along the length. Experimental results show that bond resistance of deformed rebar is notably higher than that of plain rebar (Kankam, 2003; Mo et al., 1996). Initial bond stress between plain bar and concrete develops due to the cohesion and then tend to the sliding resistance whilst in the deformed bar, the bond stress develops initially by ribs of the bar and concrete interaction.

2.5.4 Rebar diameter

Various studies were performed to specify the effect of bar diameter on bond stress. A wide range of experimental test was carried out to investigate the effect of rebar diameter on the bond stress and bar slippage. (Eligehausen et al., 1982). The results indicated that with the enhancing bar diameter the maximum bond resistance reduces slightly. But the bar diameter was not affected by bond friction significantly. Other series of experimental results indicated that the smallest bar diameter develops the highest bond stress (Alavi-Fard et al., 2004; Kankam, 2003).

2.5.5 Concrete confinement

The reinforced concrete structures with cross ties or closed link endured higher stress due to the concrete lateral extension. Experiment results show that the concrete infill ductility increase in the CFT column due to the confinement effect of the steel tube (Liang, 2009). The concrete confinement also significantly improve ductility, particularly when the maximum bond strength achieved (Alavi-Fard et al., 2004). Moreover, with a higher level of confinement, maximum ultimate frictional bond and bond resistance are increased (Eligehausen et al., 1982).

The findings indicated that confined concrete depending on materials properties tends to develop a bond between concrete and steel bar or bolt whilst unconfined concrete split and fail. In addition, concrete confinement significantly effects on the stress-strain relationship and shows higher strain and strength (Eurocode 2, 2004). Figure 2.47 illustrates the stress-strain behaviour of confined and unconfined concrete.



Figure 2.47: Stress-strain curve for confined and unconfined concrete

2.5.6 Loading rate

The bond stress-slip relationship affects the bar slippage rate (Eligehausen et al., 1982). However, The bond failure is not related to the loading rate (Ožbolt et al., 2006). The study shows that the tensile loading rates do not effect on the maximum bond stress significantly. Nevertheless, the slippage rate increases with higher loading rate Mo et al. (Mo et al., 1996). Similarly, other test was performed under different displacement rates and achieved same results Alavi-Fard et al. (Alavi-Fard et al., 2004).

2.6 Design of the bolts for structural concrete members

Nowadays, the advantages of the concrete result in to be utilised widely in different industries such as building, off-shore structures, mining and etc. Researchers are more interested in increasing the strength of the connected members. Extensive design guidelines are available for anchor bolts in ACI 318-08 (ACI 318-08, 2008) and CEB (CEB, 1990; CEB, 1994; CEB, 1997).

2.6.1 Failure modes

The failure mode and effect of load in different direction must be considered in the design of anchor bolts for concrete members. However, the following literature is focused on tensile loading. The anchor bolt strength, the embedded depth, the distance from the edge and the concrete compressive strength has affected the failure modes significantly.

Experimental studies are depicted five different types of failure mode under tensile loading or displacement as illustrated in Figure 2.48.



Figure 2.48: Failure modes of Anchor bolts

(ACI 318-08, 2008)

In a ductile failure mode, the steel bolt or connected member is yielded and inelastic strains appear before any concrete fail or breakout occurs. The nominal strength of an anchor bolt under tensile loading is defined as:

$$N_{sa} = A_s f_{ub}$$
 2.1

Where N_{sa} is the steel bolt strength, A_s is the effective cross sectional area, and f_{ub} is the ultimate stress of the steel bolt.

In the pull-out failure mode, the concrete crushing and conical formation occur above the bolt head and the head of the bolt comes upside near surface of the concrete. The existing codes do not suggest any method for ultimate pull-out failure, theoretically. However, The pullout tensile strength (N_{pn}) of a single headed steel bolt can not to be higher than the concrete crushing load value as given in Equation 2.2 (ACI 318-08, 2008).

$$N_{pn} = \psi_{c,p} \, N_p = \psi_{c,p} \, 8 \, A_{brg} \, f_c'$$

Where N_{pn} is the tensile pullout strength, $\psi_{c,p}$ is a modification factor taken as 1.0 for cracked concrete at service load levels or 1.4 for uncracked concrete, A_{brg} is the net bearing area of the steel bolt head, and f_c' is the cylinder compressive strength of the concrete. In brittle failure mode the concrete member is splitting or breakout before the steel bolt or fastened member yielding.

Theoretical design of the concrete breakout failure is a greatly important and many steel bolts are fabricated such that the concrete fails before bolt yielding.

Generally, two main extensive testing methods were performed over the last decade to design concrete breakout failure; cone method (ACI 349-06, 2006) and pyramid method (Fuchs et al., 1995).

The cone method was performed based on limited experimental results (Cannon et al., 1981) that were published in ACI 349 (ACI 349-06, 2006). Figure 2.49 (a) illustrates concrete breakout for single anchor bolt under tensile loading, based on ACI 349 (ACI 349-06, 2006). In this method, the angle between failed surface and steel bolt head is assumed to be 45° and the tensile stress on the failure cone of concrete is constant.

For a single anchor bolt that affected by confined edges or overlapped cones under tensile loading, the concrete capacity is defined as:

$$N_{cb} = 0.96 \sqrt{f_{cu}} \, d_{emb}^2 \left(1 + \frac{d_h}{d_{emb}} \right)$$
 2.3

Where N_{cb} is the concrete capacity, f_{cu} is the cube concrete compressive strength, d_{emb} is the effective embedment, and d_h is anchor bolt head diameter.

In the pyramid method (Fuchs et al., 1995) the angle between four sides of pyramid shape breakout surface and steel bolt head assume to be 35° as shown in Figure 2.49 (b). The concrete capacity is given as:

$$N_{cb} = k_{nc} \sqrt{f_{cu}} \, d_{emb}^{1.5} \tag{2.4}$$

Where N_{cb} is the concrete capacity, k_{nc} is a factor taken as 15.5 for cast-in-place concrete.

Experimental tests demonstrate that pyramid method is able to forecast concrete breakout strength with constant accuracy for all condition while the cone method prediction is unstable for a different application (Eligehausen et al., 1998). This difference was related to an embedment depth of the anchor bolt. In the cone method the failure area is increases with the square of the embedment depth, whilst in the pyramid method, the area enhances with the embedment depth to the 1.5 power. The design guidelines such as ACI 318 (ACI 318-08, 2008), British Standard (CEN/TS, 2009) and CEB (CEB, 1997) are based on pyramid method.



Figure 2.49: Concrete breakout failure (a): cone method (b): pyramid method (ACI 349-06, 2006; Fuchs et al., 1995)

2.6.2 Headed rebar reinforcement in beam to column connections

Headed reinforcement is an alternative for traditional hooked bars in reinforced concrete structures which can be difficult to bend large rebar sizes to form hooks. In addition, the application of headed-bars can reduce reinforcement in cumulative areas such as connections that complicate concrete casting. Headed bars are made from a square or circular plate which is welded to the bar end to increase bearing area and anchorage capacity of the bar. The tensile force in the bar can be anchored by a combination of bearing on the ribs and on the head, as illustrated in Figure 2.50.

Experimental studies clarify the advantages of headed bars. However, essential studies are needed to indicate the possibility of utilising headed bars in reinforced concrete structures.

A series of pull-out test with short and long-headed bar were tested under tensile loading to propose a design guideline for the headed bar application (DeVries et al., 1999). The variable parameters were a concrete strength, embedment depth, edge distances, transverse reinforcement, bar length, and head size. The results revealed that net head bearing area, edge distance, and the concrete compressive strength were effected on the failure mode.



Figure 2.50: Headed bar

Forty eight pull-out tests were conducted by Choi et al. (2002) to investigate the tensile strengths and the load-displacement relationship of headed bars compared with hooked bars. Test variable parameters were concrete strengths, reinforcing bar diameters, embedment depths, edge conditions, column reinforcement, and single versus multiple bar pullout. The results indicated that the head increased the resistance between bar and concrete effectively and results in bar yielding. Moreover, the load- displacement curve for 90° deformed bars (hook bar) was similar to headed bars.

Further pull-out tests performed to investigate the effect of the shape and thickness of the anchor head on the pullout behaviour of bars under tensile load (Park et al., 2003). The result indicates that the ultimate capacity of identical bar diameter is increased with enhancing anchor head net area but enhancing of the bar thickness is not directly effect on ultimate capacity.

Another program was performed to investigate the possibility of using headed bar in bridge fabrication (Thompson et al., 2006). The findings show that the headed bar behaviour can be divided into two steps. In the first step, the anchorage was performed by bond stress and peaked at the end of this step. In the second step, the bond stress curve starts to deteriorate and the load transferred to the bar head. In this step, anchor head bearing increased and bond stress decreased. At the end of this step, the bar was yielded or head failed. The capacity of the headed-bar including the peak bearing capacity has decreased the bond stress between the peak bar stress and anchor head.

Further pull-out tests were executed with test parameters including the head shape, size and head connecting method (welding or threading) (Kang et al., 2010). The results indicated that all shapes of heads and head-connecting methods approximately behaved in the same manner. In addition, the small size heads were significantly anchored in the beam to column connection.

The characteristics of headed bars including manufacture technology, quality control, and headed bar size are available in ASTM A970 (ASTM, 2009). ACI 318 defined the development of headed and mechanically deformed bars in tension (ACI 318-08, 2008).

The development length is the required embedment length of reinforcement to develop strength at a critical zone and define as:

$$l_{dt} = \frac{0.19 \,\psi_e \, f_y \, d_b}{\sqrt{f_c'}}$$
 2.5

Where l_{dt} is the embedment length of reinforcement, ψ_e is a modification factor taken as 1.2 for epoxy coated reinforcement and 1.0 for other types, f_y is the yield strength of the bar, d_b is the bar diameter, f'_c is the cylinder compressive strength of concrete, A_{brg} is the net bearing area of anchor head and A_b is the bar area based on bar diameter.

According to Equation 2.5, the development length is almost 80% of the development length for hooked bar

ACI 318 (ACI 318-08, 2008) defined minimum ratio of $\frac{A_{brg}}{A_b} = 4$ and the normal weight concrete mixes to achieve an accurate result from Equation 2.5.

The ratio of $\frac{A_{brg}}{A_b} < 4$ is certified the anchorage for the elastic and inelastic deformation, however the behaviour is not compatible with Equation 2.5 and required further testing to determine adequate anchorage.

2.7 Concluding remark

This chapter has presented a review of various studies relate to the CFT column connections and bond stress behaviour of the fasteners. The presentation of CFT column opened new horizons in high-rise building constructions in seismic zones (AIJ, 1997; Bergmann et al., 1995; Morino et al., 2002). However, an ideal beam-to-CFT column connection, especially for CFT bolt connections has not obtained yet. Researchers conducted various steel beam to CFT column moment connections as well as welded or bolted to achieve the required strength and stiffness for special moment resisting frames. Moreover, the proposed connections must be economical and easy for fabrication.

The studies demonstrated that through beam to column welded connection is the economical compared with existing types while satisfying the required strength and stiffness for rigid connection. Nevertheless, researchers are more interested in semi-rigid connections because of ductility and energy dissipation capacity (Jingfeng et al., 2013), easy and fast construction of these type of connections. But bolt connections are still not

being fully used due to the moment resisting connections efficiency. The bolt slippage, weak anchorage of the bolt inside the concrete core, elongation of the bolt, and column bending near bolts holes are common problems (sections 2.3). These problems prevent to obtain maximum strength capacity of the bolt and reduce the moment-resisting capacity of the connection.

The most of the existing studies were focused on the column face strengthening or using non-economic methods. Accordingly, this type of connection is almost useless especially in tall building constructions due to the lower bond strength between the bolt and the concrete, significant column face deformation and bolt slippage within the concrete core (Jingfeng et al., 2013; Tizani et al., 2013a; Tizani et al., 2013b; Wang et al., 2016). Existing blind-bolts, HB (Barnett et al., 1999) and EHB (Pitrakkos, 2012), and Extended Ajax bolt (Agheshlui et al., 2011) are adopted for on-site bolting of the beamto-CFT column connections. Nevertheless, further studies are required to solve the bolt slip problem, column face deformation and lower moment- resisting capacity. Therefore, a beam-to-CFT column bolted connection with maximum moment capacity, in consideration of the fabrication cast, is not applicable.

This study proposes a novel type of blind bolt known as TubeBolt to solve the existing problems, maximise the moment-rotation capacity and provide a ductile moment connection. The results could improve the design of the bolt connection to CFT column.

CHAPTER 3: RESEARCH METHODOLOGY

3.1 Introduction

This Chapter includes a description of the experimental test program in detail. The program is presented in three sections i.e.; (i) description of the novel Double-Sleeve TubeBolt, (ii) pull-out tests of novel TubeBolt under tensile monotonic loading and (iii) half-scale tests of moment connection with novel TubeBolt connected to CFT column under lateral cyclic loading. Moreover, fabrication process of the specimens, test setup, instrumentations, experimental tests loading procedures and relative materials tensile test properties (coupon) are discussed.

3.2 Methodology

Figure 3.1 summarises the methodology of study and relative activities in a flowchart format to achieve the objectives of this research programme.

In the first step, a novel Double-Sleeve TubeBolt was designed based on the limitation of the existing bolts and CFT bolt connection demands. Ultimately, the TubeBolt was fabricated successfully and satisfied the expected specifications. For the first time, in this study, a special bolt with different mechanical properties was proposed to fasten the beams to the CFT column. The novel bolt was passed the preliminary qualification tests to be used as an ideal fastener in CFT connections. Then, the anchorage performance of the TubeBolt and EHB such as bolt slippage, bolt elongation, column face deformation and maximum pull-out strength of specimens were evaluated and discussed through thirty monotonic tensile pull-out test programme (see section 3.2).

Finally, the pull-out result was used to propose six half-scale CFT column connections for both TubeBolt and EHB.



Figure 3.1: The flowchart of research methodology

3.3 Propose a novel TubeBolt for CFT column connections

Lack of the specific bolt for the CFT column can be clearly observed in all existing studies. The researchers were conducted various modifications and methods to enhance the efficiency of the beam to CFT bolt connection and economical properties. However, the existing results indicate that the ideal CFT bolt connection has not achieved yet. Herein, TubeBolt can provide a connection with excellent seismic performance as well as economical specifications.

3.3.1 Brief description of the proposed TubeBolt

Referring to Figures 3.2 to 3.7, the TubeBolt (100) is desinged for connecting the steel beam to CFT column or using as a strong anchor bolt in other industries. This Double-Sleeve Tubebolt is an all-in-one bolt and can be inserted through a bolt hole quickly and easily. The TubeBolt is user-friendly as it can be tightened easily similar to existing blind bolts without using specific wrenches and equipment. Referring still to Figures 3.5 to 3.7, the shank comprises threaded portion (113) formed on the outer surface (12) thereof for tightening the fastener in order to connect the structural members particularly a beam (700) and a hollow column (800) of a building as illustrated in Figure 3.7. This Tubebolt has three separate sections: a clamping body (30a), a clamping member (40), an anchoring body (30b), an anchoring member (50), and an end anchor member (60). These two bodies (40, 50) are expandable simultaneously during tightening. As shown in Figures 3.5 and 3.7, it is preferred that the clamping sleeves (43) and the anchoring sleeves (53) expand when the fastener is being tightened. The novel fastener is designed with a great shape and mechanical properties with double sleeves to enhance the anchorage performance of the Tubebolt within the limited bolt hole space. The anchor body significantly prevents sliding and elongation of the bolt during loading. The bolt shank division between the clamping sleeve, anchoring sleeve, and end anchor member can uniformly transfer the stress into the three part of the shank and reduce bolt elongation.

In the connection between the beam and the tubular column, the end-plate and hollow column are clamped together by TubeBolt, and then the hollow column is filled with concrete. The interaction, bonding, bearing between the embedded parts (the shank, the clamping sleeve, the anchoring sleeve, and end anchor member), and concrete prevent the bolt slippage, bolt elongation and column face deformation failure modes. Figure 3.7 illustrate the side view of the Tubebolt while clamping and anchoring sleeves are in the expanded mode to connect a beam (700) to a hollow column (800) filled with concrete (910).



Figure 3.2: The rear perspective view of the novel TubeBolt







Figure 3.4: The front perspective view of the novel TubeBolt



Figure 3.5: The 3D view of the TubeBolt of which its sleeves are expanded



Figure 3.6: The side view of the TubeBolt of which its sleeves are expanded



Figure 3.7: The side view of the Beam to CFT column TubeBolt connection

3.3.2 Industrial application

In light of that the anchoring member of the TubeBolt is able to significantly transfer the load or tension in terms of the beam to CFT column connection, and the segregation between the clamping member and the anchoring member on the sleeve member prevents any bolt elongation, the TubeBolt is sufficiently industrial applicable. Technical strength (load transfer capacity, ductility, stiffness and strength) of the TubeBolt aside, the CFT column has the ability to reduce pillar-per-space (square feet) ratio i.e. it increases space by reducing pillars within the space. Due to the TubeBolt's technical capabilities to load distribution and the segregation of its shank (or sleeve member) due to the position of the clamping member and the anchoring member to prevent elongation and therefore movement thereof, slippage and column surface flexibility can be prevented, and strength, ductility and moment-resisting capacity of the connection can be remarkably increased.

Experimental research indicates that the anchorage performance of the tubeBolt is 2.5 times higher than EHB. Nevertheless, the number of the bolts to connect the beam to CFT column and fabrication cost can be significantly reduced.

3.4 Monotonic tensile pull-out test

Currently, despite all shortcomings and disadvantages of EHB bolt, it is the only suitable version of a fastener for CFT column connections. Hence, steel beam to CFT column welded connections have dominated in the current constructions and bolted CFT connections is still not being fully used. The novel TubeBolt is designed to eliminate existing problems and introduce a desirable bolt to be used in CFT column connections. To evaluate the strength, stiffness, ductility and bond and anchorage mechanism of the TubeBolt in the concrete compared with EHB component, a total of 30 pull-out tests (with 10 different parameters examined) were performed under tensile monotonic loading. Minimum of three repeating testings were conducted for each arrangement to confirm the reliability of test results. The main purposes of the program are to monitor the load transfer capacity of the TubeBolt and EHB bolt component and find force-displacement relationship subjected to monotonic tensile loading.

3.4.1 Test specimens

The specimens were denoted as POT1 to POT10, where POT denotes the pull-out test. Test specimens were designed using commercial steel square hollow tube section $200 \times 200 \times 8$ mm and $200 \times 200 \times 6$ mm. The length of the all specimens was 650 mm. The main circular rigid plate diameter and thickness were 150 mm and 25 mm, respectively. All specimens were examined in a manner to observe the probable plastic behaviour of steel tube face near the bolt hole. The specimens were established so that the boundary condition was not influenced by the concrete failure mode of the pull-out test. All tested TubeBolt and EHB bolts grade were 10.9. The pull-out test specimens are summarized in Table 3.1. The following parameters were varied in the tests: Bolt type (TubeBolt or EHB), bolt diameter (16mm or 20 mm), end anchor member, hollow tube wall thickness and bolt hole diameter.

The objectives of the testing was: (1) for specimens POT1 to POT4: to study the effect of the level of confinement (confined and semi-confined) on the the TubeBolt components bond stress, load transfer capacity and anchorage properties to compare with the EHB bolt, (2) specimens POT3 to POT6: to study the effect of the end anchor member on the performance of the TubeBolt under monotonic tensile loading to compare with the EHB bolt, (3) specimens POT7 and POT8: to study the effect of the tube wall thickness on the monotonic tensile performance of the TubeBolt compare to the EHB bolt and (4) specimens POT9 and POT10: to study the effect of the bolt diameter on the monotonic tensile behaviour of the TubeBolt compare with the EHB bolt. The connection details and arrangement of the test specimens are illustrated in Figure 3.8.



Section A-A

Figure 3.8: The details and arrangement of the test specimens

				Table 3.1	l: Pull-out test sp	ecimens details				
Specimen	Bolt type	Bolt	Bolt hole	Tube wall	Concrete	Concrete tensile	Bolt	Bolt	Shank	Embedded
		diameter	diameter	thickness	compressive	strength	component	grade	length	length
		(d_b, mm)	(mm)	(mm)	strength	(f_t, MPa)			(mm)	(d_{emb}, mm)
					(<i>f_{cu}</i> , MPa)					
POT1	TubeBolt	16	100	6	41.80	2.87	Full	10.9	170	112
POT2	EHB	16	100	6	41.80	2.87	Full	10.9	170	112
РОТ3	TubeBolt	16	28	8	40.30	2.84	Full	10.9	170	112
POT4	EHB	16	28	8	41.80	2.87	Full	10.9	170	112
POT5	TubeBolt	16	28	8	40.30	2.84	Nut free	10.9	170	112
POT6	EHB	16	28	8	40.30	2.84	Nut free	10.9	170	112
POT7	TubeBolt	16	28	6	41.50	2.87	Full	10.9	170	114
POT8	EHB	16	28	6	41.50	2.87	Full	10.9	170	114
РОТ9	TubeBolt	20	35	8	41.50	2.87	Full	10.9	170	107
POT10	EHB	20	35	8	40.3	2.84	Full	10.9	170	107

Note: The concrete strength results were obtained from the average compressive strength on the day of testing.

3.4.2 Specimens fabrication

The fabrication of all specimens commenced with the preparation of the hollow tube with 650 mm length. The dimension of specimens was specified based on the maximum value of the d_{emb} to avoid the effects of the boundary condition on the pull-out test failure mode (see section 2.6.1) and find suitable place for the reaction frames installation as illustrated in Figure 3.9. Two bolt holes with a required bolt diameter (28 mm diameter for bolt size 16 and 35 mm diameter for bolt size 20) were placed at the center of the tube face and rigid plate. The third hole was also drilled in opposite face of the tube to place LVDT1 (see Figure 3.10 to 3.12). Small nuts were welded to the end of all bolt shanks to place an extended thin rod for LVDT connection (see Figure 3.14). Additionally, a thin PVC pipe was inserted through a hollow tube hole and connected to the bolt end as illustrate in Figures 3.10 to 3.12. For specimens POT1 and POT2, a plate with 100 mm diameter was cut at the centre of the tube face as illustrated in Figure 3.14. Then a bolt hole with 16 mm diameter was drilled at the center of the cut plate. Afterward, the cut plate and rigid plate were replaced on the tube face, respectively. The rigid plate was temporarily welded on tube face to facilitate tightening of the bolt and prevent concrete leakage during casting. The welding was removed after the concrete casting.



Figure 3.9: Definition of boundary condition and reaction frame situation

The manufacturing of specimens continued with bolt tightening. All bolt are tightened with a handheld torque wrench at required torque (190 Nm for bolts with 16 mm diameter and 300 Nm for bolts with 20 mm diameter) to clamp the rigid plate (with 25 mm thickness) to the hollow tube (with 8mm and 6mm thickness) face. The rigid plate is used to transfer the tensile load on the bolt head. The thickness of the rigid plate is defined such that the elastic bending of the plate was not observed throughout the test.

The rigid plate is simulated the end-plate in a beam to CFT column connection and also employed as a part of the loading frame of the setup. Figure 3.15 shows the pull-out specimen and rigid plate configuration. Thereby, total clamping thickness including tube wall thickness was 33 mm and 31 mm for square hollow tube sections $200 \times 200 \times 8$ mm and $200 \times 200 \times 6$ mm, respectively. Finally, the hollow tube was filled with concrete. All specimens involve concrete filled square tube with different thickness that simulates a CFT (CFT) column function.



Figure 3.10: Inner view of the specimen with TubeBolt and inserted PVC pipe



Figure 3.11: Inner view of the specimen with EHB bolt and inserted PVC pipe



Figure 3.12: PVC pipe hole for LVDT setup



Figure 3.13: Bolt shank with a small nut weld to the end



Figure 3.14: Semi-confined specimens in details



Figure 3.15: Clamped rigid plate configuration

3.4.3 Material properties

All the steel hollow tubes were filled with a concrete mix from the similar batch as given in Table 3.2. The bottoms of the steel hollow tubes were closed and sealed with foil to avoid concrete water evaporation and cured with a wet sponge from the top. Standard concrete cylinders were cast from the same batch as the test specimens and cured in water at a laboratory temperature of approximately 30°C. The specimens were transferred to the laboratory after fabrication and placed vertically to fill the inside of the tubes with concrete. To determine the concrete compressive and tensile strength, several standard concrete cylinder samples ($150 \times 300 \text{ mm}$) were tested. The compressive and tensile strength of the concrete was determined using the standard cylinder compression and tension test (see Figures 3.16 and 3.17). The concrete properties of pull-out specimens on the day of testing are reported in Table 3.1.

Conc	rete	Cement	Fine	Coarse	Water	Super
compressiv	e strength	(kg/m^3)	aggregate	aggregate	(kg/m^3)	plasticiser
(Mp	ba)	5	(kg/m^3)	(kg/m ³)		(%)
Nominal Actual						
40	40.70	50	106.90	122.50	25	0.50
50 51.30		50	94.10	105.90	20.60	0.50

Table 3.2: Concrete mix design

Note: The concrete strength results were obtained from the average testing at 28 days.

The tensile coupons from the steel tube, rigid plate and bolts (used in specimens) were designed and tested according to ASTM 370 (ASTM, 2012) to determine the yield stress (f_y) , ultimate stress (f_u) and elongation (d) at rupture. The results of the material tests from the steel components are presented in Table 3.3.



Figure 3.16: A view of the standard cylinder compressive test



Figure 3.17: Front view of the standard cylinder tensile test

Component	Coupon	Yield strength	Ultimate	Elongation	
		(N/mm^2)	strength	(%)	
			(N/mm^2)		
Rigid plate	Plate	332.30	476.90	25.80	
SHS 200 × 200 × 8	Plate	348.70	492	23.52	
SHS 200 × 200 × 6	Plate	341.60	488	22.73	
16 mm shank with	Rod	986	1127	9.30	
grade 10.9					
20 mm shank with	Rod	963	1113	9.10	
grade 10.9					
Bolt sleeve	Rod	379	518	23.40	

Table 3.3: Material properties of steel coupons for pull-test

Note: The results were obtained from the average of three coupon test.

3.4.4 Experimental test setup

The pull-out test setup is performed in order to evaluate the individual behaviour of TubeBolt and EHB bolt with or without end anchor under monotonic tensile loading condition. The general arrangement of the main test setup, applied loads, and reaction frames are illustrated in Figures 3.18 and 3.21. The adjustable crosshead of the Universal Testing Machine was used as a ground and the test setup was established on it to simplify the installation process. The specimens were placed in the test setup so that the bolt and CFT tube longitudinal axes were vertical and horizontal, respectively.

To distribute a uniformly tensile load in the loading frame system over the bolt head, another rigid plate with 25 mm thickness is connected to the first plate with a four-bolt system (four pieces of 20mm high strength threaded rod), in a parallel direction. Moreover, a high strength rod was placed in the centre of the upper rigid plate so that the rod longitudinal axis was vertical. The end of the high strength rod was clamped to the Tensile Testing Machine's grip as shown in Figure 3.19. To provide reaction frames, two rectangular hollow section tubes were designed with required strength, thickness and size (see Figure 3.20). Reaction frames are placed on the face at the both ends of the specimen with constant measured distance to give a sufficient space for concrete pyramid failure pattern formation (see section 2.6.1). As mentioned in section 3.2.2, suitable location for the reaction frame was found based on the maximum valued of the d_{emb} as shown in Figure 3.9.



Figure 3.18: General arrangement of the test setup



Figure 3.19: Loading frame for the test setup



Figure 3.20: Reaction frame components



Figure 3.21: A view of the experimental pull-out test setup

3.4.5 Instrumentation

Figure 3.22 shows the schematic view of the LVDT's arrangement. A load cell connected to the top of the Universal Testing Machine measured the value of the tensile load force applied to the head of the bolt. LVDT1 and LVDT2 (linear variable differential transformers) were used to measure the bolt end and the bolt head slippage, respectively as depicted in Figures 3.22 to 3.25. Strain gauges were fixed to the specimens to measure the strain response, such as the amplitude and distributions in critical areas. The relative data were recorded using data logger.



Figure 3.22: Schematic view of the LVDT locations



Figure 3.23: LVDT1 and an extended rod connected to the end of the bolt



Figure 3.24: LVDT2 fixed on the bolt head



Figure 3.25: Pull-out test instrumentations with all configurations

3.4.6 Loading procedure

All the specimens were subjected to monotonic tensile loading conditions. A constant tensile load was applied in displacement control at a rate of 1.5 mm/min up to failure (see section 2.3.6) using a 2000 kN hydraulic Universal Testing Machine on the top of the

loading frame which is connected to the testing bolt head. The applied load rate was recorded by the machine recorder system as shown in Figure 3.26.



Figure 3.26: Front view of Universal Tensile Machine system

3.5 Half-scale cyclic test of moment connection to CFT column

To determine the required seismic performance of novel Double- Sleeve TubeBolt steel beam to CFT column connection, which represents an interior moment frame connections, a series of experimental programme was conducted and the characteristics of the connection were evaluated. Six half-scale novel TubeBolt beam to column connections were tested under incremental amplitude quasi-static cyclic loading to monitor the connection behaviour, including failure modes, stiffness degradation, strength ratio, failure modes, ductility and hysteretic energy dissipation.

3.5.1 Test specimens

The specimens were denoted as NBBC1 to NBBC6, where NBBC denotes the novel blind bolt connection. Due to the limited actuator stroke, the specimens were designed to have sufficient elastic behaviour until 1% story drift. Specimens NBBC2 and NBBC4 were examined in the ductile regime to observe the behaviour of the plastic hinges on the beam near the connection. The column-beam moment ratio was calculated in such a way to investigate the effects of the TubeBolt, EHB bolt and twin stiffener plates have on the longitudinal stress and the deformation of the CFT column, bending of the extended endplate and bolt failure. The twin stiffener plates were used to move the plastic hinge far away from beam-extended end-plate welded zones.

The beam section properties are presented in Table 3.4. Test specimens NBBC1 to NBBC6 were designed using commercial steel beam section $W16 \times 5-1/2 \times 26$ with grade S275JR which was manufactured by Hyundai Steel Company.

The extended end-plate width, height and thickness were 165 mm, 694 mm and 20 mm respectively. The twin stiffener plates width, height and thickness were also 165 mm, 100 mm and 20 mm respectively. Specimens NBBC1 and NBBC5 were used bolt shank grade 8.8 with 170 mm length and others employed bolt shank grade 10.9 with similar length. The bolt connection to CFT column test specimens are summarized in Table 3.5. This arrangement was proposed to provide adequate beam strength and stiffness and to observe connection failure mode. The CFT column height and span of the beams were 1910 mm, 1450 mm, respectively. For all specimens, commercial steel hollow section (SHS) column $300 \times 300 \times 8$ was used.

The beam was selected from I-shaped hot-rolled sections. The connection details and arrangement of the test specimens are illustrated in Figure 3.27.

Table 3.4: Beam section properties								
	Specimen	section	Depth	Beam width	Web thickness	Flange thickness	Section modulus	Section area
			d (mm)	b _{bf} (mm)	t _{bw} (mm)	t _{bf} (mm)	$S_x (mm^3)$	As (mm ²)
	NBBC 1 to 6	$W16 \times 5-1/2 \times 26$	400	139.70	6.35	9.52	629263.30	4954.80

Table 3.4: Beam section properties

Table 3.5: Properties of the specimens

Specimen	Bolt type	Bolt	Bolt	Bolt hole	Bolt	Tightening	Hollow	Twin	Concrete	Concrete
		diameter	grade	diameter	component	torque	column	stiffener	compressive	tensile
		(d_b, mm)		(mm)		(N.m)	thickness	plates	strength	strength
				C			(mm)		(<i>f_{cu}</i> , MPa)	(f_t, MPa)
NBBC1	TubeBolt	16	8.80	28	Full	190	8	Yes	41.80	2.87
NBBC2	TubeBolt	16	10.90	28	Full	300	8	Yes	41.80	2.87
NBBC3	TubeBolt	16	10.90	28	Full	300	8	No	40.30	2.84
NBBC4	TubeBolt	16	10.90	28	Nut free	300	8	Yes	41.80	2.87
NBBC5	EHB	16	8.80	28	Full	190	8	Yes	40.30	2.84
NBBC6	EHB	16	10.90	28	Full	300	8	No	40.2	2.81



Figure 3.27: The connection details and arrangement of the test specimens

The following parameters were varied in the tests: bolt type, bolt grade and twin stiffener plates. The objectives of the testing was: (1) for specimens NBBC1, NBBC2 and
NBBC5: to study the effect of the bolt type, bolt grade, and twin stiffener plates on the seismic performance of the TubeBolt connection to compare with the EHB bolt connection, (2) for specimens NBBC2 and NBBC3: to study the effect of the bolt grade 10.9 and twin stiffener plates on the seismic performance of the TubeBolt connection to compare with the EHB bolt connection, (3) for specimens NBBC2 and NBBC2 and NBBC2 is study the effect of the TubeBolt connection to the effect of TubeBolt components on the seismic performance of the TubeBolt connection, (4) for specimens NBBC2 and NBBC6: to study the effect of the bolt type on the seismic performance of the connection.

3.5.2 Design of the connections

Currently, there is no available guideline for CFT column bolted connection design. However, the pull-out test results indicate that the novel TubeBolt can achieve the maximum tensile strength capacity and eliminate bolt slippage and column face deformation due to the excellent anchorage performance as demonstrated in chapter 4. Hence, the design is based upon current codes for beam to I-section column bolted connection, where it can be assumed that if the bolt sliding and column face deformation will not effect on the behaviour of the connection.

Six half-scale bolted connection specimens with steel beams and concrete filled square steel tube columns were designed based on the AISC specification for special moment-resisting frames (AISC/LRFD, 2004; ANSI/AISC 341-10, 2010; ANSI/AISC 358-10, 2010; ANSI/AISC 360-10, 2010) and on the two guidelines for designing CFT columns (ASCE, 1994; Choi et al., 2006) as well as the ACI specification (ACI 318-08, 2008). The design procedures of the connections are summarized as follows:

Due to the limited numbers of the specimens, the connections were designed such that the performance of different types of the bolt connections as well as the anchorage capacity of the bolts can be observed and compared. The maximum yield strength of the steel tube, the column thickness and size, widththickness ratio of the beam and the column and the concrete compressive strength of the composite column satisfied the AISC specification for special moment resisting frames (ANSI/AISC 341-10, 2010). The beam flexural capacity is considered as the full plastic moment of the section. The design procedures are presented in details in Appendix A.

3.5.3 Specimens fabrication

To fabricate the specimens, sixteen bolt holes were drilled on extended endplates and both sides of the steel tube where the extended endplate was located on tube faces.

The edges of the web and flanges of the beam were then beveled and welded to the extended endplate using full penetration welds. Thereafter, the extended endplates were placed on the column faces over the bolt holes. Afterward, all bolts were inserted through the bolt holes and tightened using a handheld torque wrench to clamp extended endplates to tube faces at required torque.

For three specimens, a pair of twin stiffener plates were located over the extended endplate at the top and bottom of the beam flanges.

For all specimens, two thick plates were placed, welded and stiffened on both beams ends to uniformly distribute the load of the actuators and prevent the beam ends deformation throughout the test. Finally, the hollow column was filled with concrete.

The fabrication processes for the proposed connection are shown in Figures 3.28 to 3.32.



Figure 3.28: Extended end-plate connection without twin stiffener plates



Figure 3.29: Extended end-plate connection with twin stiffener plates



Figure 3.30: Interior view of the TubeBolt connection



Figure 3.31: Connection after fabrication



Figure 3.32: Specimens after painting

3.5.4 Material properties

All the hollow columns were filled with a concrete mix from the similar batch as shown in Table 3.2. The bottoms of the CFT columns were closed and sealed with foil to avoid concrete water evaporation and cured with a wet sponge from the top. Standard concrete cylinders were cast from the same batch as the test specimens and cured in water at a laboratory temperature of approximately 30°C. The specimens were transferred to the laboratory after fabrication and placed vertically to fill the inside of the tubes with concrete. To determine the concrete compressive and tensile strength, several standard concrete cylinder samples $(150 \times 300 \text{ mm})$ were tested. The compressive and tensile strength of the concrete was determined using the standard cylinder compression and tension test. The compressive and tensile strength of the concrete was determined using the standard cylinder compression and tension test (see Figures 3.16 and 3.17). The concrete properties of pull-out specimens on the day of testing are reported in Table 3.1. The tensile coupons from the steel tube, bolts and plates (used in beams and extended endplates) were designed and tested according to ASTM 370 (ASTM, 2012) to determine the yield stress (f_y), ultimate stress (f_u) and elongation (d) at fracture. The results of the material tests from the steel components are presented in Table 3.6.

3.5.5 Experimental test setup

The general arrangement of the main test setup, applied loads, and reaction frames are illustrated in Figure 3.33. The specimens were placed in the test setup so that the column and girder longitudinal axes were vertical and horizontal, respectively. To distribute a uniformly axial load over the concrete and steel tube, rigid steel column caps were used for the top and bottom of the CFT column. Each column end was fitted inside of the cap using a steel wedging plate to prevent any movement between the cap and the column ends.

To simulate a pinned connection for the column ends, steel rollers were welded to the end caps. The column caps were supported on the loading plane using high-strength threaded rods. The ends of these rods were connected to the caps using a special tool known as a swivel and strong support. The swivels were allowed to fully rotate the specimens on the loading plane. The rods were preloaded at the end to prevent any lateral movement of the specimen. A strong frame also supported the column caps, which was restrained from the strong floor by the lateral threaded rods. The column was laterally supported near the loading point and the restrained end by special bearings that were attached to the strong frame to keep the specimen stable and to hold the 2000 kN actuator in the correct position throughout the test. Moreover, the beams were laterally supported in the loading point and the plastic hinge to prevent lateral torsional buckling.

Component	Coupon	Yield strength (N/mm ²)	Ultimate strength (N/mm ²)	Elongation (%)
W 16 × 5-1/2 × 26	Flange	288.47	426.50	28.60
	Web	291.31	426.75	26.76
SHS 300 × 300 × 8	Plate	357.60	496.25	24.04
Endplate	Plate	335.70	467.30	25.80
16 mm shank with grade 10.9	Rod	986	1127	9.30
16 mm shank with grade 8.8	Rod	808.70	1009.30	12.20
Bolt sleeve	Rod	379	518	23.40

Table 3.6: Material properties of steel coupons for moment connections



Figure 3.33: Test setup arrangement

The maximum spacing of the flange edge from the lateral support was nearly 0.9 m and was based on AISC seismic provisions (ANSI/AISC 341-10, 2010). Figures 3.34 to 3.43 show the main components of the test setup.



Figure 3.34: Bottom support of specimen



Figure 3.35: Side view of steel column cap for column ends



Figure 3.36: Roller at the bottom of the column caps



Figure 3.37: Swivels connected to steel column caps



Figure 3.38: Axial loading setup and threaded rod installation



Figure 3.39: Beam supports near loading point, side B



Figure 3.40: Beam supports near loading point, side A



Figure 3.41: Beam supports near beam end



Figure 3.42: Another view of the test setup



Figure 3.43: Side view of the test setup frame

3.5.6 Instrumentation

Figures 3.44 to 3.50 are illustrated the instruments arrangement. The bolts displacements were recorded using linear variable differential transformers (LVDT) to monitor the bolts slippage. The steel beams and steel tube displacements were measured to compute the rotation of the connections. Two LVDTs were measured the column cap displacement to monitor the test setup stability during the test. Two horizontal LVDTs, which were located at the both ends of the extended end plate at the top and bottom of the flanges and connected to the rigid frame, recorded the relative rotation of the beam versus the joint to determine the rigidity of the connection. Moreover, the diagonal LVDTs recorded the shear behaviour of the panel zone area. All the specimens were painted to track the deformation process in detail during the loading history procedure.

Strain gauges were fixed to the specimens to measure the strain response, such as the amplitude and distributions in critical areas of connection. The beam web and flange surfaces in the vicinity of the plastic hinge, the column face near the connection, and extended endplate face near the bolts were the main locations for the strain gauges. The strain data were recorded using data loggers.



Figure 3.44: LVDTs to record the bolt displacement



Figure 3.45: LVDTs arrangement



Figure 3.46: Diagonal LVDTs to record the shear behaviour of the panel zone



Figure 3.47: Strain gauges arrangement on the beam flange and web



Figure 3.48: Strain gauges arrangement on the column



Figure 3.49: Strain gauges arrangement on the panel zone



Figure 3.50: Strain gauges arrangement on the extended end-plate

3.5.7 Quasi-static cyclic loading

All the specimens were subjected to two loading steps. In the first step, a constant axial load was applied using a 2000 kN hydraulic actuator on the top of the column as a reaction force from the upper floors. The axial load value for all specimens was 20% of the maximum nominal compressive strength capacity (P_{no}) of the column. Afterwards, two dynamic actuators with a 500 kN capacity using isolated pumps applied uniform vertical loads on the beam ends in opposite directions according to the loading procedure proposed by the AISC provisions (ANSI/AISC 341-10, 2010) to simulate deformation in a building subjected to lateral loads. A similar displacement was simultaneously controlled for both sides to simulate seismic loading in small increments until specimen. Figure 3.51 and Table 3.7 show the loading history for the test specimens. The beam end displacement was recorded by the actuators, which were directly connected to the data logger.



Figure 3.51: Loading history

Steps	Drift	Number of	Displacement	Speed rate	Frequency
		cycles	(mm)	(mm/min)	
	(%)				
1	0.375	6	5.54	10	0.007668
2	0.5	6	7.39	20	0.0111
3	0.75	6	11.08	30	0.01127
4	1	4	14.78	30	0.008457
5	1.5	2	22.17	30	0.005638
6	2	2	29.56	30	0.004228
7	3	2	44.34	30	0.00282
8	4	2	59.12	30	0.0021143
9	5	2	73.90	30	0.001691
10	6	2	88.68	30	0.001409

Table 3.7: Loading history specification for beam to CFT column connections

3.6 Concluding remark

The novel TubeBolt is designed to eliminate existing problems and introduce a desirable bolt to be used in CFT column connections. 30 pull-out tests were performed under tensile monotonic loading. Additionally, six half scale bolt connections under cyclic loading were conducted. Monotonic pull-out tests and beam to CFT column bolt connection under cyclic loading arrangement, fabrication process, test setup assembly, material properties of test specimens and instrumentations were reported.

The chapter was presented Novel Double- Sleeve TubeBolt for CFT columns, drawing, benefits, efficiency and industrial application.

Pull-out test specimens were designed using commercial steel square hollow tube section. Two bolt holes with a required diameter were placed at the center of the tube face, rigid plate and opposite face of the tube. Small nuts were welded to the end of all bolt shanks to place an extended thin rod. For two specimens, a plate with 100 mm diameter was cut at the centre of the tube face. The fabrication proceeds with bolt tightening. All bolt are tightened to the face of the hollow section tube along with the rigid plate. Finally, the hollow tube was filled with concrete. Specimens involve concrete filled square tube with a different thickness that simulates a CFT column function. The Universal Testing Machine has measured the value of the tensile load force applied to the head of the bolt. LVDT1 and LVDT2 measure the bolt end and the bolt head slippage, respectively. Strain gauges were placed to the specimens to record the strain response. Specimens were subjected to a uniform tensile load.

Beam to CFT column bolt connection specimens were designed using commercial Ishaped hot-rolled steel beam section W16 \times 5-1/2 \times 26. For all specimens, commercial steel hollow section (SHS) column was used. Sixteen bolt holes were drilled on extended endplates and both sides of the steel tube where the extended endplate were located on tube faces. The edges of the web and flanges of the beam were welded to the extended endplate. Thereafter, the extended endplates were bolted to the column face. For three specimens, a pair of twin stiffener plates were located over the extended endplate at the top and bottom of the beam flanges. Finally, the hollow column was filled with concrete.

The bolts and column cap displacements, the steel beams and steel tube rotations and the shear behaviour of the panel zone were recorded using linear variable differential transformers (LVDT). All the specimens were painted to track the deformation process in detail during the loading history procedure. Strain gauges were fixed to the specimens to measure the strain response in critical areas.

All the specimens were subjected to two loading steps. In the first step, a constant axial load was applied on the top of the column. Afterward, two dynamic actuators applied uniform vertical loads on the beam ends in opposite to simulate deformation in a building subjected to lateral loads.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Introduction

This chapter describes the findings of experimental in detail. The findings are presented in two sections; (i) novel Double-Sleeve TubeBolt and existing EHB pull-out tests under tensile monotonic loading and (ii) novel Double-Sleeve TubeBolt and existing EHB moment connections to CFT column under lateral cyclic loading. The performance of pull-out specimens are presented by means of load-slip relationship. The performance of all test specimens are merely discussed. In addition, the result of novel double-sleeve TubeBolt pull-out test specimens as well as novel double-sleeve TubeBolt connections, are compared with the relative EHB specimens to exhibit the significant effect of the new proposed TubeBolt on the CFT moment connection specimens are also investigated in terms of hysteretic moment-rotation relationship. The seismic performance of the connections are evaluated, including the failure modes, hysteretic performance, rotation capacity, strength degradation, rigidity, ductility, and energy dissipation. The experimental findings will contribute to expanding the database of moment-resisting beam to CFT column connections.

4.2 Experimental pull-out test results

The previous pull-out test with EHB employed a rigid steel hollow tube with 20 mm wall thickness to eliminate tube face bending under monotonic tensile load (Pitrakkos et al., 2013). Hence, the evaluation of the bolt anchorage performance such as bolt sliding within the concrete core was almost impossible. Despite the weak anchorage mechanism of the EHB, the pull-out test results of the EHB is not a helpful database for a moment-resisting beam to CFT column bolt connections design. Since the CFT column with 20 mm thickness is not economical in the building construction industry, the pull-out tests

in this study employ the steel tubes with 8 mm and 6 mm thickness to simulate an actual behaviour of the bolt in CFT column connections.

4.2.1 Test observations and failure modes

The specification of each specimen such as bolt type, bolt components, bolt size, bolt grade, and tube wall thickness is given in Table 3.1. In order to calculate the elongation of the both EHB and TubeBolt in similar status, the TubeBolt elongation is also computed up to a maximum strength of the EHB specimen.

4.2.1.1 Specimen POT1

The semi-confined concrete filled tube surface of type 16 mm diameter TubeBolt with end anchor member after failure is presented in Figure 4.1.

When the TubeBolt bolt is subject to tensile load, the applied load distributes within the connecting member through concrete-end anchor, concrete-double expanded sleeve and concrete-shank bonds. With an increase in the applied load, the stress is uniformly distributed through its concrete-end anchor, concrete- double expanded sleeve, and concrete-shank bonds. Consequently, the majority of stresses transfer into the concrete with minimum concrete crushing and splitting crack prior to failure at the maximum load and results in the TubeBolt strongly surrounded by concrete at the end of the test. Thus, the conical nut, expanded sleeve and end anchor nut of the TubeBolt components tends to pull out and breakout the semi-confined concrete surface prior to its ultimate state. Finally, the maximum load capacity of the bolt was sharply reduced in a failure state. This sudden load reduction represents the higher potential energy which was released through concrete failure. In addition, the segregation of the bolt shank between the clamping sleeve, anchoring sleeve, and end anchor member can uniformly distribute the load into the different part of the shank and prevents significant bolt elongation. In addition, the bolt achieved its 83.5% of maximum tensile strength, 147.5% higher load carrying capacity and 47.06% lesser elongation in comparison with specimen POT2 as presented in Table 4.1.



Figure 4.1: The concrete face of the semi-confined specimen POT1 after failure

The results depicted that the notable loading capacity is depicted by the type semiconfined TubeBolt specimen with end anchor member and failure mode of type TubeBolt included a minor bolt elongation, concrete splitting crack and breakout at higher applied load. This indicates the effect of the anchorage mechanism in the absence of the tube surface and with respect to the tensile load distribution; the anchor system of the TubeBolt is remarkably capable to distribute the applied load into the concrete core as depicted by the load-slip relationship that is shown in Figure 4.2.



Figure 4.2: Load-slip relationship of specimen POT1

Specimen	Bolt type	Maximum Load	Bolt slippage up to the max. strength of		Bolt elongation	Remark
		(kN)	the EHB specimen (mm)		(mm)	
			Bolt head	Bolt end		
		F _{max}	δ_t	δ_{end}	Δ_e	
POT1	TubeBolt	171.20	0.34	0.25	0.09	Semi-confined
POT2	EHB	69.15	2.78	2.61	0.17	Semi-confined
РОТ3	TubeBolt	205.04	1.48	1.38	0.10	Full bolt
POT4	EHB	115.56	3.42	3.11	0.31	Full bolt
POT5	TubeBolt	186.69	0.97	0.89	0.08	End anchor free
POT6	EHB	79.56	22.77	22.50	0.27	End anchor free
POT7	TubeBolt	187.38	0.75	0.65	0.10	6mm steel tube
POT8	EHB	95.12	2.48	2.14	0.34	6mm steel tube
РОТ9	TubeBolt	293.17	1.15	1.08	0.07	20 mm bolt
POT10	EHB	124.34	34.70	34.47	0.23	20 mm bolt

Table 4.1: Maximum load, slippage and elongation of the bolts

Note: The results were obtained from the average of three similar tests

4.2.1.2 Specimen POT2

The semi-confined concrete filled tube surface of type16 mm diameter EHB with end anchor member is illustrated in Figure 4.3. When the EHB bolt is subject to tensile load, the applied load is distributed within the connecting member through its concrete-end anchor, concrete-expanded sleeve, and concrete-shank bonds. With an increase in the applied load, the stress is concentrated on the concrete-end anchor small contact area. Consequently, the end anchor member significantly crushes and splits its surrounded concrete and slide freely within the concrete. Thereby, the conical nut, expanded sleeve and end anchor nut of the EHB tend to pull out and breakout the semi-confined concrete surface at lower load prior to its ultimate state. Finally, the maximum load capacity was reduced quickly through the test. This quick load reduction represents the severe concrete core crushing. The failure mechanism of type semi-confined EHB with end anchor nut is the significant concrete core crushing, splitting crack and concrete face breakout. Moreover, the bolt achieved 33.7% of the maximum tensile strength of the bolt as presented in Table 4.1.



Figure 4.3: The concrete face of the semi-confined specimen POT2 after failure

The results indicate that the least loading capacity is depicted by the specimen POT2 and failure mode of type EHB included a major bolt elongation, concrete severe crushing, concrete splitting crack and breakout at lowest applied load. This indicates that the end anchor nut of the EHB is not capable to distribute the applied load effectively into the concrete core as depicted by the load-slip relationship that is shown in Figure 4.4.



Figure 4.4: Load-slip relationship of specimen POT2.

4.2.1.3 Specimen POT3

The concrete filled tube surface of type16 mm diameter TubeBolt with end anchor member is depicted in Figure 4.5.



Figure 4.5: The tube face of the confined specimen POT3 after bolt rupture

Apart from the observations mentioned in section 4.2.1.1, the load is also distributed within tube face-expanded sleeve-conical nut contact area. However, the stress in this area was relieved due to the excellent anchorage performance, which has eliminated tube face flexibility and reduced the bolt slip. It is thus the transition of the load within the concrete core via anchorage mechanism omits the concrete splitting crack and breakout which was observed in type EHB. Finally, the bolt was ruptured and the test was terminated. However, the bolt achieved its maximum tensile strength, 77.4% higher load carrying capacity and 68% lesser elongation in comparison with specimen POT4 as presented in Table 4.1. It was observed that the failure mode of type TubeBolt involved a minor bolt elongation and bolt rupture at higher load, minor crack at the concrete surface and negligible sliding as illustrated in Figure 4.6 and 4.7. Most importantly, the tube face deformation was not observed at maximum applied load while the anchorage mechanism was fully developed. This indicates that the double-expanded sleeves member along with the end anchor of the TubeBolt is capable to distribute the applied load significantly into the concrete core as depicted by the load-slip in Figure 4.8. If the second expanded sleeves are attached to the bolt, the pull-out behaviour can obtain a stiffer load-slip response of type TubeBolt.



Figure 4.6: The concrete face of the confined specimen POT3 after bolt rupture



Figure 4.7: Tube face and rigid plate at maximum load of specimen POT3



Figure 4.8: Load-slip relationship of specimen POT3

4.2.1.4 Specimen POT4

Figure 4.9 shows the specimen type 16 mm diameter EHB with end anchor member at the end of the test.



Figure 4.9: Specimen POT4 at the end of the test

In addition to all the observations mentioned in section 4.2.1.2, the load is also distributed within tube face-expanded sleeve-conical nut contact area. As mentioned earlier, the stress concentration on concrete-end anchor contact area. Thus, the end anchor crushes and splits its surrounded concrete and slide easily inside the concrete. Hence, the load directly transfer to the conical nut-bolt hole contact zone. The conical nut of the EHB tends to push through the bolt hole clearance and deform the tube face prior to its ultimate state. Finally, the expanded sleeves were widen the bolt hole and the maximum load carrying capacity maintained almost constant through the test. The majority of this constant load involves the strength of the hollow tube surface. In addition, the bolt achieved 56% of the maximum tensile strength of the bolt as presented in Table 4.1. It was found that the failure mode of type EHB included a tube surface deformation at lower load, bolt elongation and subsequently, concrete splitting crack and breakout at the end of the test as shown in Figure 4.9. This indicates that the end anchor of the EHB is not capable to distribute the applied load effectively into the concrete core as depicted by the load-slip relationship that is shown in Figure 4.10.



Figure 4.10: Load-slip relationship of specimen POT4

4.2.1.5 Specimen POT5

Figure 4.11 illustrates the specimen type16 mm diameter TubeBolt without end anchor member at maximum load.



Figure 4.11: Specimen POT5 at maximum load

Apart from the observations mentioned in sections 4.2.1.1 and 4.2.1.3, the end anchor member was removed. The bolt has achieved its 91% of maximum tensile strength, 134.6% higher load carrying capacity and 70% lesser elongation in comparison with specimen POT6 as presented in Table 4.1.

It was observed that the failure mode of type TubeBolt without end anchor member involved a negligible bolt elongation, sliding at higher load, and minor concrete crack. Most importantly, the tube face deformation was not observed at maximum applied load. Once again, this indicates the effect of the anchorage mechanism in the absence of end anchor member with respect to the of the tensile load distribution; double-expanded sleeves members of the TubeBolt is capable to distribute the applied load significantly into the concrete core as depicted by the load-slip relationship that was shown in Figure 4.12.



Figure 4.12: Load-slip relationship of specimen POT5

4.2.1.6 Specimen POT6

Figure 4.13 illustrates the specimen type16 mm diameter EHB without end anchor member at maximum load.



Figure 4.13: Specimen POT6 at maximum load

In addition to all the observations mentioned in sections 4.2.1.2 and 4.2.1.4, the end anchor member was removed. With an increase in the applied load, the stress was only concentrated on the concrete-expanded sleeve contact area near the concrete surface. Consequently, the concrete face has easily cracked and splitted by the expanded sleeve and the bolt slide freely within the concrete. Hence, the total stress directly transfer to the conical nut-bolt hole contact zone. Finally, the expanded sleeves have widen the bolt hole and the maximum load was obtained after significant slippage. This maximum load represents the strength of the hollow tube surface. Furthermore, the bolt has achieved its 38.80% of maximum tensile strength as presented in Table 4.1. It was found that the failure mode of type EHB included a major bolt elongation and sliding, tube surface bending at lower load and subsequently, concrete splitting crack and breakout. This indicates that the EHB without end anchor member is unable to distribute the applied load into the concrete core as depicted by the load-slip relationship that was shown in Figure 4.14.



Figure 4.14: Load-slip relationship of specimen POT6

4.2.1.7 Specimen POT7

Figure 4.15 shows the specimen type16 mm diameter TubeBolt with end anchor member and 6 mm thickness of steel hollow tube at maximum load.



Figure 4.15: Specimen POT7 at maximum load
In addition to all the observations mentioned in section 4.2.1.3, the maximum load carrying capacity of the bolt was lower than that of specimen POT3. Moreover, the bolt has achieved its 91.4% of maximum tensile strength, 97% higher load carrying capacity and 70% lesser elongation in comparison with specimen POT8 as presented in Table 4.1. Hence, the failure mechanism of type TubeBolt with 6 mm thickness of steel hollow tube is negligible elongation and sliding, minor cracks at maximum load as depicted by the load-slip relationship that is illustrated in Figure 4.16.



Figure 4.16: Load-slip relationship of specimen POT7

4.2.1.8 Specimen POT8

Figure 4.17 shows the specimen type16 mm diameter EHB with end anchor member and 6 mm thickness of steel hollow tube at the end of the test.

Apart from the observations mentioned in section 4.2.1.4, the maximum load carrying capacity of the bolt was lower than that of specimen POT4. Furthermore, the bolt has achieved its 46.39% of maximum tensile strength as presented in Table 4.1. The load-slip relationship of specimen POT8 is shown in Figure 4.18.



Figure 4.17: Specimen POT8 at the end of the test



Figure 4.18: Load-slip relationship of specimen POT8

4.2.1.9 Specimen POT9

Figure 4.19 shows the specimen type 20 mm diameter TubeBolt with end anchor member at maximum load.



Figure 4.19: Specimen POT9 at maximum load

Apart from the observations mentioned in sections 4.2.1.3, the specimen POT9 achieved higher tensile load capacity. The bolt has achieved 88.57% of its maximum tensile strength, 135.78% higher load carrying capacity and 69.56% lesser elongation in comparison with specimen POT10 as presented in Table 4.1.

It was observed that the failure mode of type TubeBolt was involved a minor bolt elongation, 8 mm sliding at maximum load of 293.2 kN, and concrete cracks. Most importantly, the effective column face deformation was not observed at maximum applied load. This indicates that the TubeBolt anchorage mechanism is significantly capable to distribute the higher applied load of 20 mm bolt into the concrete core as depicted by the load-slip relationship that as shown in Figure 4.20.



Figure 4.20: Load-slip relationship of specimen POT9

4.2.1.10 Specimen POT10

Figure 4.21 shows the specimen type 20 mm diameter EHB with end anchor member at maximum load.



Figure 4.21: Specimen POT10 at maximum load

In addition to the observations mentioned in sections 4.2.1.4, the specimen POT10 exhibited higher column face deformation.

When the EHB bolt is subject to tensile load, the applied load is distributed within the connecting member through its concrete-end anchor nut, concrete-expanded sleeve and tube face-expanded sleeve-conical nut contacts. With an increase in the applied load, the stress is concentrated on concrete-end anchor nut contact area.

It was found that the failure mode of type EHB included a major bolt elongation and sliding, significant tube surface bending at maximum load and subsequently, concrete crushing, splitting cracks and breakout. In addition, the bolt has achieved its 37.56% of maximum tensile strength as presented in Table 4.1. The load-slip relationship is illustrated in Figure 4.22.



Figure 4.22: Load-slip relationship of specimen POT10

4.2.2 Effect of tested parameters

To investigate the individual behaviour of the bond and anchorage function of the TubeBolt and EHB components, a series of experimental tests were conducted. The main target of the tests was to prepare a suitable and adequate experimental test arrangement. The objectives of this section include investigating the effect of concrete confinement and contribution of the bolt components on the anchorage performance. The detail of the specimens is summarised in Table 3.2.

The effects of various test parameters on the load-slip relationship of the TubeBolt and EHB are presented in this section. These results compare the behavior of the TubeBolt and EHB components with that achieved by the main parameter specimens, permitting for key findings to be designed.

4.2.2.1 Effect of concrete semi-confinement

The specimens POT1 and POT2 were designed in a manner so that the expanding sleeves contact mechanism with tube face was eliminated. This was carried out by opening a larger bolt hole (100 mm diameter) than that of defined for the special bolt diameter (28 mm).

To evaluate the components bond and anchorage mechanism of the bolts, pull-out tests were performed on TubeBolt and EHB that were embedded directly through the larger bolt hole in the concrete filled tube without any contact with steel tube.

The 100 mm diameter clearance could eliminate interaction between bolt and tube wall to fully transfer the tensile load to the embedded bolt components. Such design permits the applied tensile load to be transferred by the embedded component of the bolt into the concrete. The effect of concrete semi-confinement on the load-slip relationship of the TubeBolt and EHB is presented in Figure 4.23. The results was obtained from the average of three similar tests to obtain the accurate results.



Figure 4.23: Effect of the semi-confinement on TubeBolt and EHB

When the TubeBolt with full component was tested in semi-confined concrete with 8mm tube wall thickness, excellent improvements are noted. It is found that the least resistance is provided by that of the EHB semi-confined concrete with 8mm tube wall thickness. The performance of both TubeBolt and EHB with full components was much similar up to 30 kN. With increasing tensile load, the strength, stiffness and ductility characteristics of the TubeBolt were maintained compared to EHB. In specimen POT1, the majority of stresses were uniformly transferred into the concrete due to the excellent anchorage performance. In specimen POT2, the minority of stresses was transferred into the concrete due to the weak anchorage and results in significant concrete damage and strength reduction.



Figure 4.24: Load-slip relationship of the semi-confined and confined specimens

As mentioned earlier, the interaction between bolt and tube wall was eliminated interaction in semi-confined specimens and embedded directly in concrete to fully transfer the tensile load to the embedded bolt components. Due to the elimination of tube wall in the vicinity of the bolts in specimens 1 and 2, highest level of the stress were transferred into the concrete core compare to the other specimens. Hence the anchorage performance of the TubeBolt and EHB can be clearly clarified.

Figure 4.24 compares the load-slip behaviour of semi-confined and confined TubeBolt and EHB components in similar condition. The graphs indicate the weak anchorage performance in type EHB in the semi-confined condition compares with type TubeBolt confined specimens. In contrast, the load-slip relationship of confined TubeBolt illustrates a significant modification in comparison with the semi-confined TubeBolt specimen. The graph indicates that the TubeBolt in semi-confined condition could achieve higher strength capacity.

Overall, it is demonstrated that the strength and ductility of the bolts are related to their mechanical properties. However, the type EHB is significantly affected by the concrete semi-confinement.

4.2.2.2 Effect of tube wall thickness

Figures 4.25 and 4.26 present the effect of increasing the tube wall thickness from 6 mm to 8 mm for both TubeBolt and EHB specimens, respectively. The ductility, stiffness, yield and ultimate strength of the component are slightly enhanced in comparison with 6mm tube wall thickness. In contrast, the yield and ultimate strength of both bolts relate to the type of bolt, demonstrating that the inherent anchorage mechanism of the bolts is independent of the tube wall thickness. The test results indicate that the ductility of the bolts is also affected by the alteration of tube wall thickness. However, the failure mode of TubeBolt specimens was varied; from bolt rupture to the negligible sliding (including bolt elongation) and minor cracks at maximum load.

However, the failure mode of EHB specimens was not changed and similar to the 8mm tube wall specimen, the test was failed by concrete splitting and tube face bending near bolt hole.

Basically, it is clarified that the strength and ductility of the both bolts are corresponded with their mechanical characteristics that are affected by the tube wall thickness as shown in Figure 4.27.



Figure 4.25: Effect of tube wall thickness on TubeBolt specimens



Figure 4.26: Effect of tube wall thickness on EHB specimens



Figure 4.27: Effect of tube wall thickness on TubeBolt and EHB specimens

4.2.2.3 4.1.5 Effect of anchor elements

To assess the contribution of the end anchor element of the TubeBolt and *EHB* component, experimental tests were conducted on bolts with and without end anchor

element under confined concrete conditions. The TubeBolt is compared with the *EHB* to conclude on the contribution of end anchor and double-sleeve elements. The tests will demonstrate whether end anchor member, double-sleeve member or bond is the essential contributor of the stress within the concrete.

The experimental test results are shown in Figures 4.28 to 4.30. The graphs demonstrate that the lowest resistance and ductility is provided by that of the *EHB* without end anchor member, with a standard shank length of 170 mm. When the end anchor assembled to the EHB, the stiffness and load capacity are sharply enhanced due to the increased bond and anchorage resistance supplied by the threaded shank and end anchor member that interacts with the concrete core as shown in Figures 4.28.



Figure 4.28: Effect of the end anchor member on EHB specimens

However, the amplitude of the modified EHB performance with increasing bond is not comparable to that observed by the *TubeBolt* which does not include an end anchor member as presented in Figures 4.29. The load-slip relationships of the 16 mm TubeBolt specimen indicate that the anchorage mechanism is fully developed.



Figure 4.29: Performance of TubeBolt and EHB without end anchor member

This demonstrates that the double-sleeve component of the TubeBolt is the main contributor in the load transfer within the bond and anchorage mechanism.

Figures 4.30 compare the specimens TubeBolt with and without end anchor element. When the TubeBolt involves end anchor, the stiffness and maximum load capacity are slightly enhanced (9% increase) due to the increased bond and anchorage mechanism provided by both double-expanded sleeve and end anchor members.



Figure 4.30: Effect of the end anchor member on TubeBolt specimens

It indicates that the end anchor member of the TubeBolt is the subsidiary contributor in the load transfer within the bond and anchorage mechanism in TubeBolt in comparison with EHB.

4.2.2.4 Effect of bolt diameter

Figure 4.31 compares the load–slip relationship of the TubeBolt and EHB in consideration of 20 mm bolt diameter, with an enhanced tightening torque (See Table 3.5). It is found that the initial stiffness of the TubeBolt component is sharply enhanced compare with EHB, and as anticipated, the load is also maintained to a much higher load. The yield and ultimate strength, as well as ductility increase in TubeBolt. Once again, these modifications correspond with mechanical performance of the bolt, which in this experiment are related to the double-expanded sleeve anchor mechanism.



Figure 4.31: Performance of TubeBolt and EHB with 20 mm diameter

The failure mode of the 20 mm TubeBolt specimen involved the yielding, minor sliding and eventual concrete cracks at maximum load of 293.7 kN. Meanwhile, the failure mode of the 20 mm EHB specimen involved the yielding and severe sliding, concrete splitting, breakout and tube face drastic deformation after maximum load of

124.34 kN. Figure 4.32 compares the load–slip relationship of the TubeBolt component in consideration of 16 and 20 mm bolt diameters.



Figure 4.32: Performance of TubeBolt with 16mm and 20 mm diameter

It indicates that the initial stiffness of the TubeBolt component is significantly enhanced in the case of the 20 mm bolt diameter, and as predicted, it is also maintained by a much higher force in comparison with 16 mm EHB.

The TubeBolt yield and ultimate strength, as well as ductility increase sharply with the increase in bolt diameter. Once again, these effects relate to the significant anchorage mechanism of the bolt, which is directly dependent upon the diameter and double-sleeve member.



Figure 4.33: Performance of EHB with 16mm and 20 mm diameter

On the contrary, the results indicate that the initial stiffness of the EHB component is enhanced insensible and its yield point is significantly reduced in the case of the higher bolt diameter. However, it is almost maintained to same load capacity that was achieved with 16 mm diameter EHB bolt as shown in Figure 4.33. This phenomenon can be directly related to the weakness of anchorage and load transfer mechanism of the EHB components in consideration of 20 mm bolt diameter.

Closer inspection of the load-slip relationships of the 16 mm and 20 mm TubeBolt specimens (Figure 4.32) indicates that the anchorage mechanism is almost fully developed by means of 20 mm TubeBolt. In contrast, the load-slip graph of the 16 mm and 20 mm EHB specimens (Figure 4.33) indicates that both specimens are almost failed in similar ultimate load (115.56 kN and 124.34 kN, respectively) and the anchorage mechanism is not fully developed due to severe bolt slippage, concrete crushing, splitting and breakout.

Overall, the Tubebolt yield and ultimate strength, as well as ductility are significantly enhanced with the increase in bolt diameter compare with EHB as presented in Figure 4.34. This performance corresponds to the mechanical properties of the TubeBolt components, which in this case depend on the diameter of the bolt.



Figure 4.34: Effect of bolt diameter on TubeBolt and EHB specimens

However, the use of 20 mm diameter and grade of 10.9 is also feasible within type TubeBolt, achieving enhanced stiffness, strength, and ductility characteristics in comparison with 16 mm TubeBolt of the same grade. Conversely, The EHB yield, ultimate strength, and ductility are almost consistent with the increase in bolt diameter. This behaviour relates to the mechanical properties of the EHB components, which are independent of the diameter of the bolt. Hence, the application of EHB with 20 mm diameter does not effect on the performance.

4.2.2.5 Contribution of anchor components

Closer examination of the load-slip relationships of the specimens POT1, POT2, POT3 and POT5 shows that the contribution of first expanded sleeve including a threaded shank, second expanded sleeve and end anchor components in type 16 mm TubeBolt grade 10.9 on the anchorage mechanism were 31.39%, 59.61% and 9%, respectively. These

proportions can be extended to the 20 mm diameter TubeBolt with the same specifications due to the similarities.

The contribution of type EHB components in load transmission is quite various due to the different anchorage mechanism, as the double-sleeve mechanism in type TubeBolt is the active anchor member compared with that of EHB with end anchor active member. However, the 16 mm EHB mechanism can achieve only 40.39% of the maximum strength of the bolt grade 10.9 while 59.61% of the bolt strength was useless due to the slippage within the concrete core.

4.3 Experimental Beam to CFT column connections results

4.3.1 Test Observations and failure modes

4.3.1.1 Specimen NBBC1

An initial flexural yielding was observed in the second cycle of the 1.5% story drift. Symmetrical paint flaking in the inner part of the flanges was detected (approximately 50 mm from the connection). At 2% story drift, the amplitude of the flexural yielding was extended and paint flaking extended slightly to the outer part of the flanges. At 3% story drift, flange local buckling was noted in the beams. Paint flaking spread to approximately 140 mm at the top flange and 110 mm at the bottom flanges along the beam length and continued to the web near the beam–web intersection. At the end of the second quarter of the second cycle of the 4% story drift, slight beam lateral movement and flange local buckling was observed and the botts near bottom flange was reached to maximum strength capacity and ruptured and the test was terminated. In addition, inelastic bending was observed on the extended end-plate due to the ruptured and elongated bolt. Similar observations were recorded on both sides of the beams. Column face deformation near bolt holes and cracking in the welded area were not observed. The failure modes of the test specimen at 4% story drift is shown in Figures 4.35 and 4.36.



Figure 4.35: Beam flange deformation in specimen NBBC1



Figure 4.36: Bolt rupture without column face deformation in NBBC1

4.3.1.2 Specimen NBBC2

An initial flexural yielding was observed in the second cycle of the 1.5% story drift. Symmetrical paint flaking in the inner part of the flanges was detected (approximately 50 mm from the connection). At 2% story drift, the amplitude of the flexural yielding was extended and paint flaking extended slightly to the outer part of the flanges. At 3% story drift, flange local buckling was noted in the beams. Paint flaking spread to approximately 150 mm at the top flange and 120 mm at the bottom flanges along the beam length and continued to the web near the beam-web intersection. At the end of the 4% story drift, slight beam lateral movement, extensive flange local buckling, and initial web local buckling were observed. At 5% story drift, flexural yielding extended approximately 150 mm along the beam length, and paint flaking spread to the inner part of the web. In addition, low cycle flange fatigue cracks were detected near the buckling area at the end of the second cycle of the 5% story drift. The test was terminated at the end of the 6% story drift due to the limited stroke of the actuators. At the end of the test, flexural yielding extended approximately 200 mm along the beam length, paint flaking spread approximately 300 mm at the flanges and 200 mm on the centre line of the web, beam lateral movement was approximately 15 mm, and the amplitude of the beam and web local buckling and beam global buckling were extensively amplified. The buckling depth on the centre line of the web was 30 mm. The buckling depth was measured in the plastic hinge area, which was approximately 150 mm from the column face. Similar observations were recorded on both sides of the beams. The concrete damage, concrete crushing, or concrete crack was not observed in the vicinity of the bolts and panel zone as shown in Figures 4.39 and 4.40. Finally, yielding, local buckling on the extended end-plate, column face deformation near bolt holes and cracking in the welded area were not observed. The failure modes of the test specimen at 6% story drift is shown in Figures 4.3 to 4.40.



Figure 4.37: Specimen NBBC2 after test



Figure 4.38: Plastic hinge without bolt rupture and column face deformation



Figure 4.39: Concrete face of specimen NBBC2



Figure 4.40: Panel zone in specimen NBBC2

4.3.1.3 Specimen NBBC3

The behaviour of the specimen was approximately similar to the observations mention in section 4.3.1.2 up to 4% story drift. At the end of the first quarter of the second cycle of the 4% story drift, the bolts near bottom flange were reached to maximum strength capacity and ruptured. Thereafter, the test was terminated. In addition, inelastic bending was observed on the extended end-plate near ruptured bolt due to the elongation of the bolt. The failure modes of the test specimen at 4% story drift is shown in Figures 4.41 and 4.42.



Figure 4.41: flange deformation in specimen NBBC3



Figure 4.42: Flange deformation and bolt rupture in specimen NBBC3

4.3.1.4 Specimen NBBC4

The performance of specimen was approximately similar to the observations mention in section 4.3.1.2. The failure modes of the test specimen at 6% story drift is shown in Figures 4.43 and 4.44.



Figure 4.43: Beam deformation in specimen NBBC4



Figure 4.44: Plastic hinge without bolt rupture and column face deformation

4.3.1.5 Specimen NBBC5

Apart from the observations mentioned in section 4.3.1.1, an initial column face deformation near upper bolt holes was observed at the second cycle of 3% story drift. At the end of the second quarter of the second cycle of the 4% story drift, the test was terminated due to the significant column face deformation in the vicinity of the bolt holes. The concrete was significantly damaged in the vicinity of the bolts as shown in Figurev4.48. However, concrete crack was not observed in panel zone (see Figure 4.49). The failure modes of the test specimen at 4% story drift is shown in Figures 4.45 to 4.49.



Figure 4.45: Beam after test in specimen NBBC5



Figure 4.46: Column face deformation in specimen NBBC5



Figure 4.47: Column face deformation and end-plate separation in NBBC5



Figure 4.48: Concrete face of specimen NBBC5



Figure 4.49: Panel zone in specimen NBBC5

4.3.1.6 Specimen NBBC6

An initial column face flexibility near upside bolt holes and bolt sliding were observed at the first cycle of 3% story drift and quickly extended up to the end of the test. At 5% story drift, the crushed concrete was clearly starting to emit from bolt holes clearance. Flexural yielding, local buckling and paint flaking on the beam web and flanges or inelastic bending of extended end-plate were not observed through the test. At the end of the test, the amplitude of the column face deformation and bolt sliding were extensively amplified. Finally, the test was terminated at the end of 6% story drift due to the significant column face deformation behind the extended end-plate. The concrete was significantly damaged in the vicinity of the bolts as shown in Figure 4.53. Moreover, concrete crach was not observed in panel zone as illustrated in Figure 4.54. The failure modes of the test specimen at 6% story drift is shown in Figures 4.50 to 4.54.



Figure 4.50: Beam after test in specimen NBBC6



Figure 4.51: End-plate separation in specimen NBBC6



Figure 4.52: Column face deformation in specimen NBBC6



Figure 4.53: Concrete face of specimen NBBC6



Figure 4.54: Panel zone in specimen NBBC6

4.3.2 Moment–rotation hysteretic curves

The characterization of the beam to column connections is represented in the momentrotation relationship. For each story drift increment, the rotation of the connection is calculated by subtracting the column rotation from the beam rotation. The hysteretic moment–rotation curves for all the specimens are illustrated in Figure 4.55.



Figure 4.55: Hysteretic moment-rotation relationship of test specimens

The moment-rotation curves exhibited linear elastic responses in the first story drift. Afterward, the stiffness in the connection gradually degraded, and the connection behaviour proceeded inelastically in the subsequent story drifts. The connections NBBC2 and NBBC4 reached a maximum rotation of 68 mrad and the test were terminated at the end of 6% story drift prior to beam flange fracture due to the limited stroke of the actuators (total of 200 mm stroke). The specimen NBBC6 also reached a maximum rotation of 68

mrad but the test stopped due to the significant bolt slippage and severe column face deformation in the vicinity of the bolt holes.

The rest of specimens reached a maximum rotation of 45.30 mrad and failed in the drift 4%. Rest of specimens were terminated because of ruptures in the bolt. Specimens NBBC5 and NBBC6 indicate significant strength and stiffness degradation in the same cycles of subsequent story drifts due to the bolt slippage and column face deformation. Moment-rotation relationship for specimen NBBC3 indicates an interruption at the mid of the first quarter of the second cycle of the 4% story drift and that can be related to the bolt rupture and releasing higher potential energy accumulated in ruptured bolts.

For specimens NBBC1, NBBC2, and NBBC3 and NBBC4, the pinching effect was not observed. On the contrary, the hysteretic curve for specimens NBBC5 and NBBC6 exhibited severe pinching and load deterioration after 3% story drift, which indicated bolt sliding and deformation of the column face and consequently a reduction of moment resisting capacity of the connections. This behaviour may be related to weak anchorage mechanism, concrete crushing and splitting around embedded EHB bolt.

In the large story drifts, the area enclosed by the hysteretic loops and the moment capacity of the connection has increased and decreased, respectively. This phenomenon may be due to the local buckling, permanent deformation of the beam flange and column face, or the beam web, concrete crushing and beam plastic hinge formation in the vicinity of the connection.

For specimens NBBC1 and NBBC5, the TubeBolt and EHB grade 8.8 with twin stiffener plates were used, respectively. The moment-rotation relationship of specimen NBBC5 indicated moment reduction and lower seismic performance in comparison with NBBC1. This can be related to the EHB sliding, concrete crushing and column face deformation of specimen NBBC5. The results indicate the effect of the bolt type on anchorage performance of the connection even with lower bolt grade.

Specimens NBBC2 and NBBC4 with a TubeBolt grade 10.9 indicated a high moment capacity, large hysteretic loops, greater energy dissipation, and better seismic performance than specimens NBBC6 with EHB grade 10.9. This indicates the effect of the anchorage mechanism with respect to the load carrying capacity; the anchor system of the TubeBolt is remarkably capable to distribute the applied load into the CFT column.

Specimens NBBC2 and NBBC4 with a TubeBolt grade 10.9 exhibited a high moment capacity, large hysteretic loops, greater energy dissipation, and better seismic performance than specimens NBBC3 with the similar bolt. This relates to the twin stiffener plates at the top and bottom of the flange. In specimen NBBC3, when the beam end subject to the cyclic loading, both flanges bend the extended end-plate at the distance of the bolts. This deformation is linear elastic in the first story drifts. Gradually, the bending behaviour proceeded inelastically in the last story drifts. Thereby, beam level arm increase due to the bending of the extended end-plate and further stress transfer to the flange-extended end-plate welded zones and bolt heads. Ultimately, the bolts rupture because of higher load on the bolt heads and plastic hinge formation near welded regions. When the twin stiffener plates is used in specimens NBBC2 and NBBC4, the thickness of the extended end-plate in critical places increased and elastic bending of the extended end-plate reduced at the top and bottom of the flange. Hence, the twin stiffener plates can effectively move the plastic hinge far away from beam-extended end-plate welded zones and the connection exhibit ductile and stable behavior throughout the experimental test. This indicates that thickness of extended end-plate significantly effects on the seismic performance of the connection. The twin stiffener plates can remarkably reduce the application of steel in bolt connections. Despite using a thin extended end-plate in connections, the twin stiffener plates could increase the thickness of thin extended endplate merely in critical regions. Overall, the proposed connection may be used in momentresisting frames in seismic zones.

4.3.3 Results and discussion

4.3.3.1 Moment-rotation hysteretic envelope curves

The moment–rotation hysteretic envelope curves for all specimens are plotted in Figures 4.56 to 4.58. The main parameters of the connection, such as the moment capacity, rotation capacity, yield point, and initial stiffness for both the negative and positive directions are presented in Table 4.2.



Figure 4.56: Moment-rotation hysteretic envelope curve (NBBC1 and NBBC5)



Figure 4.57: Moment–rotation hysteretic envelope curve (NBBC2 and NBBC4)



Figure 4.58: Moment-rotation hysteretic envelope curve (NBBC3 and NBBC6)

Table 4.2: The main parameters of the connection							
Specimen	Maximum moment		Maximum rotation		Yield point		Initial stiffness
	Moment (kN.m)	Rotation (mrad)	Moment (kN.m)	Rotation (mrad)	Moment (kN.m)	Rotation (mrad)	(kN.m/mrad)
	M _{max}	$\theta_{M.max}$	$M_{\theta.max}$	$ heta_{max}$	My	θ_y	K _i
NBBC1+	224.23	33.59	180.94	44.63	192.17	18.33	10.83
NBBC1-	222.52	33.64	193.33	44.40	187.61	15.75	11.13
NBBC2+	254.69	33.61	179.23	67.37	207.81	18.6	16.23
NBBC2-	242.26	33.51	173.78	67.20	203.51	21.25	15.93
NBBC3+	286.15	44.95	271	44.95	243.18	21.33	12.41
NBBC3-	266.93	44.30	266.93	40.80	240.64	25.12	12.46
NBBC4+	264.87	45.01	171.74	67.43	232.13	21.85	12.87
NBBC4-	240.79	44.74	149.97	67.26	212.86	16.38	15.66
NBBC5+	190.27	44	112.63	44	170.80	21.28	8.31
NBBC5-	209.28	33.07	201.032	43	165.52	16.33	9.52
NBBC6+	213.83	35.016	171.15	67.80	190.89	22.50	8.53
NBBC6-	140.08	67.30	135.01	67.60	119.91	10.12	10.38

Table 4.2: The main parameters of the connection

Note: sign '+' and '-' represent 'positive and negative direction, respectively

The initial stiffness (K_{in}) of the connections was determined from the initial slope of the hysteresis curves during the first cycle of the preliminary story drift. The rotation capacity (θ_{max}) was defined from the maximum rotation of the connection prior to failure. The moment capacity (M_{max}) was specified from the maximum moment of the connection.

Specimens NBBC2 and NBBC4 could withstand the maximum rotation of 68 mrad (6% story drift) without any rapture in bolts or flanges, with a maximum strength reduction of 20%, which satisfies the 3% story drift provision for special moment frames (ANSI/AISC 360-10, 2010), 20% strength reduction until 4% total story drift for special moment resisting frames (ANSI/AISC 341-10, 2010), and the total 4% story drift before a remarkable strength reduction or total 6% story drift before full strength dissipation (FEMA 350 SAC Joint, 2000). The rotation could exceed 68 mrad but the test was terminated due to the limited stroke of the actuators. Despite the maximum rotation of 68 mrad for specimen NBBC6, the column face significantly deformed at the end of the test. The rest of the specimens satisfy the 3% story drift provision for special moment frames, 4% total story drift for special moment resisting frames and the total 4% story drift before a remarkable strength reduction. Despite maximum rotation of 68 mrad for specimen NBBC6, the column face significantly deformed at the end of the test.

For the connections with bolt grade 8.8 and twin stiffener plates (NBBC1 and NBBC5), the maximum strength capacity and initial stiffness of specimen NBBC1 (TubeBolt) were enhanced by 10.57% and 18.85%, respectively, compared to specimen NBBC5 (EHB).

For the connections with TubeBolt grade 10.9, the initial stiffness of specimen NBBC2 (with TubeBolt and twin stiffener plates) increased 22.7%, compared to specimen NBBC3 (with TubeBolt, without twin stiffener plates)

For the connections with TubeBolt grade 10.9 and twin stiffener plates, the initial stiffness of specimen NBBC2 (with end anchor member) increased 11.32%, compared to specimen NBBC4 (without end anchor member).

For the connections with different bolt type grade 8.8 and twin stiffener plates (NBBC3 and NBBC6), the maximum strength capacity and the initial stiffness of specimen NBBC3 (with TubeBolt grade 10.9 and without twin stiffener plates) increased by 36.01% and 23.97%, respectively, compared to specimen NBBC6 (with EHB grade 10.9 and without twin stiffener plates).

4.3.3.2 Rigidity evaluation and classification of the connections

To determine the rigidity of the connection in the moment–rotation envelope curve, the stiffness of the connections was specified according to the ASCI 360 (ANSI/AISC 360-10, 2010) stiffness index value. This value is expressed as K_sL/EI , where $K_s = \frac{M_s}{\theta_s}$ is the secant stiffness of the connection at the service loads, *L* is the beam length, and *EI* is the beam bending rigidity, where M_s and θ_s are moment and relative rotation between the column and beam end, respectively.

The yield moment and rotation of the connection was determined according to the FEMA 356 pre-standard (FEMA-356, 2000). The nonlinear hysteretic envelope moment–rotation relationship was replaced with an idealized bilinear relationship with an initial slope (K_e) and a post-yield slope (α) to compute the effective lateral stiffness and yield moment, as shown in Figure 4.59. The yield rotation (θ_y) was specified from the intersection of the secant slope of the actual envelope curve at 60% of the yield moment (M_y). Thus, the area enclosed by the bilinear curve is approximately equal to that enclosed by the envelope curve at an equivalent rotation of the maximum moment (θ_{M-max}), and balances the area above and below the curve.


Figure 4.59: The idealized moment-Rotation curve

Connection stiffness is divided into three classes: fully restrained or rigid, partially restrained, and simple. If $K_sL/EI \ge 20$, then the connection is considered to be fully restrained. If $K_sL/EI \le 2$, then the connection is a simple one. If it is between this range, then the connection is considered partially restrained. The stiffness of the specimens was determined from two horizontal i.e. LVDT5 and LVDT6 that recorded the relative beam-column rotation. The LVDTs were installed horizontally on the tip of the extended end-plate face, at the top and bottom of the flanges (for specimens NBBC3 and NBBC6) or on the tip of the twin stiffener plates (NBBC1, NBBC2, NBBC4 and NBBC5). Figures 4.60 to 4.62 illustrate the rigidity of the connection in the positive and negative directions. The connections NBBC1, NBBC2, NBBC3 and NBBC4 could be classified as fully restrained (rigid). The connections NBBC5, and NBBC6 could be classified as partially restrained (semi-rigid). However, connections NBBC2, NBBC3 and NBBC4 with a TubeBolt had higher rigidities than NBBC1, NBBC5 and NBBC6, respectively.



Figure 4.60: Rigidity classification of the connections NBBC1 and NBBC5



Figure 4.61: Rigidity classification of the connections NBBC2 and NBBC4



Figure 4.62: Rigidity classification of the connections NBBC3 and NBBC6

4.3.3.3 Strength ratio

The strength ratio is one of the most important parameters used to characterize the seismic proficiency of a connection subjected to cyclic loading. It is used to evaluate strength deterioration (ATC, 1992) and is defined as:

$$R_s = \frac{M_{max}^i}{M_y},$$
4.1

where M_{max}^{i} is the maximum moment at the i-th cycle of the story drift and M_{y} is the yield moment of the connection.

The evaluation of the strength ratios demonstrated that the specimens with the same arrangement exhibited similar behaviour, as shown in Figures 4.63 to 4.65. However, the specimen NBBC6 illustrated sharp degradation compare to Specimen NBBC3. This effect relates to their various mechanical properties and significant concrete rushing and flexibility of the column face of specimen NBBC6. Moreover, Figure 4.63 Indicate strength reduction for specimen NBBC5 at last story drift due to the column face deformation and concrete crushing. Strength improvements were observed for both specimens NBBC2 and NBBC4 until failure, with NBBC2 having the highest improvement while specimen NBBC4 shows slight strength degradation due to the lack of end anchor member. This finding emphasizes the positive effect of TubeBolt anchorage properties, twin stiffener plates, end anchor member, and bolt grade. In addition, significant strength degradation in the higher story drifts was detected in specimen NBBC6 (EHB grade 10.9) for all the story drifts.



Figure 4.63: Strength degradation of the connections NBBC1 and NBBC5



Figure 4.64: Strength degradation of the connections NBBC2 and NBBC4



Figure 4.65: Strength degradation of the connections NBBC3 and NBBC6

4.3.3.4 Ductility

Ductility is another important parameter used to evaluate the seismic efficiency of a connection subjected to cyclic loading. Ductility is the ability of the structural members to withstand large plastic deformation under tensile stress without remarkable strength deterioration.

Connection ductility can be evaluated using a ductility ratio (μ) based on the envelope moment–rotation hysteretic curve, which is defined (ATC, 1992) as:

$$\mu = \frac{\Delta\theta}{\theta_y},$$
 4.2

where $\Delta\theta$ is the failure rotation and θ_y is the yield rotation of the connection. The ultimate moment or failure moment (M_u) is defined as $M_u = 0.8M_{max}$, where M_{max} is the maximum moment of the connection (ANSI/AISC 360-10, 2010), as shown in Figure 4.59.

The ductility of the connections in the positive and negative directions are presented in Table 4.3.

specimen	Ductility (μ)	specimen	Ductility (μ)
NBBC1+	2.43	NBBC4+	3.62
NBBC1-	2.82	NBBC4-	4.45
NBBC2+	4.47	NBBC5+	2.07
NBBC2-	3.66	NBBC5-	2.57
NBBC3+	3.10	NBBC6+	2.07
NBBC3-	2.76	NBBC6-	2.29

Table 4.3: Ductility of the connections

Note: sign '+' and '-' represent 'positive and negative direction, respectively.

As expected, specimen NBBC2 with the twin stiffener plates and end anchor member of the TubeBolts grade 10.9 exhibited slightly higher ductility than the specimen NBBC4 with the twin stiffener plates and without end anchor member. This behaviour can be attributed to the lesser effect of the end anchor member on the ductility of the connections. Moreover, specimen NBBC2 obtained the highest ductility between the specimens.

The lesser ductility was belonged to the NBBC6 with EHB and without twin stiffener plates due to the significant bolt sliding and concrete crushing and severe column face deformation in the vicinity of the bolt holes.

Specimen NBBC1 with the twin stiffener plates and full component TubeBolts grade 8.8 indicated a higher ductility than the specimen NBBC5 with full component EHB in similar condition. This performance can be attributed to the effect of the anchorage mechanism of the different bolts and result in concrete crushing, bolt sliding and bolt column face deformation in specimen NBBC5.

Specimen NBBC3 with full component TubeBolts grade 10.9 and without twin stiffener plates and indicated a higher ductility than the specimen NBBC6 with EHB in

similar condition. Once again, this behaviour can be attributed to the effect of the anchorage mechanism of the different and result in significant concrete crushing, bolt sliding and bolt column face deformation in specimen NBBC6.

Despite the similarity of specimens, the twin stiffener plates and different types of the bolt were used to investigate the effect of TubeBolt and twin stiffener plates on rigidity, ductility, and energy dissipation of the connections. As mentioned earlier, the connections were designed so that the behaviour of the different type of bolts and effect of the twin stiffener plates can be observed in similar condition. Hence, due to the excellent anchorage performance of the TubeBolt (see section 4.2), the connection can be designed more ductile than the existing EHB bolt connections.

However, the results indicated that the rigidity of the connections significantly influenced the ductility of the test specimens. The average ductility ratios for specimens NBBC1 to NBBC6 were 2.62, 4.04, 2.93, 4.03, 2.32 and 2.18, respectively, thus satisfying the AISC 360 specification (ANSI/AISC 360-10, 2010) and the high and medium ductility class for the moment-resisting dissipative structural frame according to the plastic rotation capacity (Eurocode, 2004). As noted in section 4.2.1.2, the testing of the connections NBBC2 and NBBC4 were not continued due to the limited stroke of the actuators. Thereby, ductility of these specimens could exceed 5, based on ductility classification.

However, the ductility ratios of the connections clearly emphasized that the bolt type, bolt grade and twin stiffener plates could significantly influence the connection ductility for an identical column size. Ultimately, it can be concluded that the introduced novel Tubebolt beam to CFT column connections have significantly higher ductility.

4.3.3.5 Energy dissipation

Structural members should have the capacity to dissipate energy under cyclic loading. One of the methods to measure dissipated energy is by calculating the area enclosed using hysteresis loops. Another way to evaluate the capacity of the material in terms of energy dissipation is by determining the equivalent viscous damping ratio (ξ_{eq}) (Chopra, 2006; Priestley et al., 1996), which is defined as:

$$\xi_{eq} = \frac{E_i}{4\pi(E_e)_i},\tag{4.3}$$

where E_i is the energy dissipated by the structural members in the *i*-th loading cycle, and $(E_e)_i$ is the elastic potential energy accumulated by an equivalent linear elastic system when the maximum displacement is reached in a static position in the *i*-th cycle.

The energy dissipation in *i*-th cycle can be calculated from the determined hysteresis loops. The elastic potential energy accumulated by an equivalent linear system $(E_e)_i$ is determined by the area under the load - displacement curve obtained in the cyclic tests. It is given by the area under a right triangle as follows:

$$(M_m)_i = \frac{|(P_{max})_i| + |(P_{min})_i|}{2},$$
4.4

$$(E_e)_i = \frac{(P_m)_i (\Delta_{max})_i}{4\pi (E_e)_i},$$

$$4.5$$

where $(P_m)i$ is the average peak load of the relative cycle, and $(\Delta_{max})i$ is the maximum positive displacement measured in the test.

Figures 4.66 to 4.68 illustrate the accumulated dissipated energy of the connections under cyclic loading. The results show that the specimens exhibit higher energy dissipation with an increase in rotations.



Figure 4.66: Accumulated dissipated energy of specimens NBBC1 and NBBC5



Figure 4.67: Accumulated dissipated energy of specimens NBBC2 and NBBC4



Figure 4.68: Accumulated dissipated energy of specimens NBBC3 and NBBC6

The total energy dissipation potential and accumulated dissipated energy of specimens NBBC1 is higher than specimen NBBC5 at the ultimate strength of the connection before bolt rupture as depicted in Figure 4.66.

In addition, the total energy dissipation of specimens NBBC2 and NBBC4 are significantly higher than other specimens at the ultimate strength of the connection as shown in Figure 4.67. As mentioned above, the experimental testing of the specimens NBBC2 and NBBC4 were terminated due to the limited stroke of the actuators. Hence, dissipated energy of these specimens could significantly enhance in story drift 7%.

Despite severe column face deformation and concrete crushing of connection NBBC6, it has obtained higher energy dissipation than connection NBBC3. The testing of the specimen NBBC3 was not continued at the second cycles of 4% story drift due to the bolt rupture. However, specimen NBBC3 indicates much higher energy dissipation up to 4% story drift compare to specimen NBBC6.

Meanwhile, specimen NBBC5 exhibited lower energy dissipation capability than the other specimens. This behavior can be attributed to the bolt type and bolt grade of specimen NBBC5.

Overall, it can be concluded that the proposed novel Tubebolt beam to CFT column connections have significantly higher energy dissipation.

4.3.4 Effect of tested parameters on beam to CFT column connections

To investigate the seismic behaviour of beam to CFT connection, a series of experimental tests were conducted. The main target of the tests was to prepare a suitable and adequate experimental test arrangement. The objectives of this section include investigating the effect of bolt type, bolt grade, end anchor element, and twin stiffener plates on the seismic performance of the connections.

The effects of various test parameters on the load-slip relationship of the TubeBolt and EHB are presented in this section. These results compare the seismic response of the TubeBolt and EHB connections.

4.3.4.1 Effect of the bolt grade

For specimens NBBC1 and NBBC2, the TubeBolt grades 8.8 and 10.9 were used, respectively. The observations and moment-rotation relationship of specimen NBBC1 indicated bolt rupture, moment reduction and lower seismic performance in comparison with NBBC2. This can be related to the lower strength of the bolt with grade 8.8 and bolt elongation.

For the connections with the TubeBolt grade 10.9 and twin stiffener plates (NBBC2), the maximum strength capacity and the initial stiffness of specimen enhanced compared to specimen NBBC1.

Specimen NBBC2 with TubeBolts grade 10.9 indicated a higher ductility than the specimen NBBC1 in similar condition. The average ductility ratios for specimens NBBC1 to NBBC2 were 2.62 and 4.04, respectively.

The total energy dissipation of specimens NBBC2 is much higher than specimen NBBC1 at the ultimate strength of the connections.

For specimens NBBC5 and NBBC6, the EHB bolt grades 8.8 and 10.9 were used, respectively. The observations and moment-rotation relationship of specimen NBBC5 indicated moment reduction and lower seismic performance in both specimens. This can be related to the lower anchorage performance of the EHB bolts, bolt sliding, concrete crushing and column face deformation. However, the initial stiffness of specimen NBBC6 enhanced compared to specimen NBBC5.

Moreover, the specimens NBBC1 and NBBC2 could be classified as fully restrained (rigid) while the connections NBBC5, and NBBC6 could be classified as partially restrained (semi-rigid).

4.3.4.2 Effect of the bolt type

For specimens NBBC1 and NBBC5, the TubeBolt and EHB grade 8.8 were used, respectively. The observations and moment-rotation relationship of specimen NBBC5 indicated column face deformation, moment reduction and lower seismic performance in comparison with NBBC1. The results indicate the effect of the bolt type on anchorage performance of the connection even with lower bolt grade.

For the connections with the bolt grade 8.8 (NBBC1 and NBBC5), the maximum strength capacity and the initial stiffness of specimen NBBC1 with TubeBolt increased compared to specimen NBBC5 (EHB).

Specimen NBBC1 with full component TubeBolts grade 8.8 indicated a higher ductility than the specimen NBBC5 with EHB in similar condition. The average ductility ratios for specimens NBBC1 and NBBC5 were 2.62 and 2.32, respectively.

The accumulated dissipated energy of specimens NBBC1 is higher than specimen NBBC5 at the ultimate strength of the connection before bolt rupture of specimen NBBC5 as shown in Figure 4.66

For specimens NBBC2 and NBBC6, the TubeBolt and EHB grade 10.9 were used, respectively. The observations and moment-rotation relationship of specimen NBBC6 indicated severe column face deformation and bolt slippage, moment reduction, pinching on moment-rotation curve and lower seismic performance in comparison with NBBC2. The results indicate the significant effect of the bolt type on anchorage performance of the connection with higher bolt grade.

For specimens NBBC2 and NBBC6 grade 10.9, the maximum strength capacity and the initial stiffness of specimen NBBC2 with TubeBolt remarkably enhanced compared to specimen NBBC6 (EHB).

Specimen NBBC2 with full component TubeBolt grade 10.9 indicated a much higher ductility than the specimen NBBC6 with EHB in similar condition. The average ductility ratios for specimens NBBC2 and NBBC6 were 4.04 and 218, respectively.

The accumulated dissipated energy of specimens NBBC2 is effectively higher than specimen NBBC6 at the end of 6% story drift.

This performance can be attributed to the effect of the anchorage mechanism of the different bolts and result in concrete crushing, bolt sliding, and column face deformation in specimen NBBC6.

4.3.4.3 Effect of the end anchor element

Specimens NBBC2 and NBBC4 could withstand the maximum rotation of 68 mrad (6% story drift) without any bolt rupture or fracture on flanges. The initial stiffness of specimen NBBC2 with end anchor member increased 11.32%, compared to specimen NBBC4 without end anchor member. However, the specimens NBBC2, NBBC4 could be classified as fully restrained (rigid).

Strength improvements were observed for both specimens NBBC2 and NBBC4 up to failure. Meanwhile, NBBC2 having the highest improvement while specimen NBBC4 shows slight strength degradation due to the lack of end anchor member. This finding emphasizes the positive effect of TubeBolt anchorage properties.

Specimen NBBC2 with end anchor member exhibited slightly higher ductility than the specimen NBBC4 without end anchor member. This behaviour can be attributed to the lesser effect of the end anchor member on the ductility of the connections. Moreover, specimen NBBC2 with end anchor member obtained the highest ductility between the specimens. The average ductility ratios for specimens NBBC2 and NBBC4 were 4.04 and 4.03, respectively. However, the testing of the connections NBBC2 and NBBC4 were not continued due to the limited stroke of the actuators. Hence, ductility of these specimens could exceed 5, based on ductility classification.

In addition, the energy dissipation of specimens NBBC2 and NBBC4 are significantly higher than other specimens at the end of the test.

4.3.4.4 Effect of the twin stiffener plates

For the specimens with TubeBolt grade 10.9, the initial stiffness of specimen NBBC2 with TubeBolt and twin stiffener plates, increased 22.7%, compared to specimen NBBC3 with similar bolt without twin stiffener plates.

The results indicated that the twin stiffener plates also effectively affect the failure mode. The connections with twin stiffener plates (specimens NBBC2 and NBBC4) exhibited a high moment capacity, large hysteretic loops, greater energy dissipation, and better seismic performance than specimens NBBC3 with the similar TubeBolt without twin stiffener plates. However, the connections NBBC2, NBBC3 and NBBC4 could be classified as fully restrained (rigid).

Specimens NBBC2 and NBBC4 with twin stiffener plates and TubeBolts grade 10.9 indicated a higher ductility than the specimen NBBC3 with similar TubeBolt. The average ductility ratios for specimens NBBC2, NBBC3, and NBBC4 were 4.04, 2.93, and 4.03, respectively. This performance can be attributed to the effect of twin stiffener plates at the top and bottom of the flange.

4.4 Concluding remark

In this chapter, total of 30 experimental pull-out tests was conducted under tensile monotonic loading. Moreover, six half-scale experimental bolt connections under cyclic loading were performed using TubeBolt and EHB. Monotonic pull-out tests and beam to CFT column performance such as the failure modes, cyclic performance, ductility, strength, stiffness, and energy dissipation of the connections were presented, evaluated and discussed. The content is summarised as follows.

The semi-confined concrete pull-out test strength capacity, ductility and mechanical properties of the TubeBolt and EHB is evaluated and discussed. In addition, the contribution of anchorage component on ductility and strength capacity and elongation investigated and the main contributor of each bolt considered. Moreover, the effect of the bolt diameter on TubeBolt and EHB performance such as ductility and strength capacity and elongation examined. Additionally, the effect of the TubeBolt on the failure modes of the pull-out test specimens evaluated.

The chapter was continued with the evaluation of half-scale beam to CFT column connections results, in this section, The rigidity of experimental TubeBolt and EHB beam to CFT connections classified and the load transfer capacity of TubeBolt and EHB beam connections studied. The failure modes of the TubeBolt and EHB beam connections were also discussed.

In addition, the effect of the various parameters such as, bolt grade, bolt type, bolt components, and twin stiffener plates on the behaviour of the connections like plastic hinge formation examined. Overall, the cyclic performance, moment-resistance capacity, ductility, strength, stiffness, and energy dissipation of the novel TubeBolt and EHB beam connections investigated.

CHAPTER 5: CONCLUSIONS

5.1 Introduction

A novel blind bolt known as Double-Sleeve TubeBolt was presented in this study. Several experimental test specimens were performed to evaluate the behaviour of novel double-sleeve TubeBolt. Thirty pull-out specimens consisting of confined and semiconfined concrete TubeBolt and EHB were performed under monotonic tensile loading. The pull-out performance of the specimens was evaluated, including the failure modes, load-slip relationship, load carrying capacity, anchorage mechanism, slippage and elongation. Furthermore, six half-scale specimens containing the proposed novel doublesleeve TubeBolt and EHB moment connection to CFT column were tested under cyclic loading. The seismic performance of the connections was evaluated, including the failure modes, hysteretic performance, rotation capacity, strength degradation, ductility, and energy dissipation. The experimental findings will contribute to expanding the database of the moment-resisting beam to CFT column connections.

5.2 Conclusions

5.2.1 Proposal of novel TubeBolt

The Tubebolt is capable of achieving the maximum anchoring inside the concrete to solve the critical semi-rigid connection problems. For these reasons, the ultimate strength of the Tubebolt can be fully achieved.

The efficiency, the load carrying capacity, dynamic properties like energy dissipation, ductility, stiffness and strength of the bolt connection are increased remarkably using the improved Double-Sleeve Tubebolt:

The TubeBolt is capable to solve the bolt's common problems such as slippage and column face flexibility problem in seismic moment-resisting CFT column connection. In this experiment the bolt diameter, bolt grade, column wall thickness, and concrete compressive strength were 16 mm, 10.9 mm, 8 mm and 50 MPa, respectively. Moreover, the Tubebolt elongation is at least 68% reduced compare with existing EHB bolt. In addition, the load carrying capacity of the Tubebolt is increased more than 2.5 times compare with existing EHB bolt. The result obtained from the average pull-out test with concrete grade 40 MPa.

5.2.2 Behaviour of TubeBolt under monotonic pull-out test

The experimental test results demonstrate that the strength and ductility of the TubeBolt and EHB bolts are related to their mechanical properties.

The evaluation of anchorage component demonstrates that the double-sleeve member of the TubeBolt is the main contributor in the load transfer within the bond and anchorage Mechanism compare to the EHB with main end anchor member. Meanwhile, the anchorage performance of the EHB was significantly lower than that of TubeBolt. This demonstrates that the double-sleeve member is the essential contributor of the load within the concrete.

The pull-out results reveal that the initial stiffness, strength capacity, yield point, as well as ductility of TubeBolt with 16 mm diameter and 10.9 grade is sharply enhanced compare with EHB and the pull-out behaviour can obtain the firmly stiffer load-slip response. It is also found that the most effective ratio of EHB bolt strength capacity was useless in all specimens due to the limited anchorage performance.

5.2.3 Parameters affecting the TubeBolt performance

The experimental pull-out test program demonstrates that the strength and ductility of the TubeBolt and EHB bolts are significantly affected by the concrete semi-confinement and tube wall thickness. The lower strength capacity is provided by that of the EHB semiconfined concrete with 6mm tube wall thickness. When the TubeBolt with full component was tested in semi-confined concrete with 8mm tube wall thickness, excellent improvements are noted.

The pull-out test results reveal that the TubeBolt significantly affects the failure mode of the specimens. The specimens with a TubeBolt (specimens POT1, POT3, POT5, POT7 as well as POT11) indicate a larger capacity and higher ductility compared to the EHB.

In addition, the tube bolt with 20 mm diameter and grade 10.9 maintained to a much higher force in comparison with 16 mm EHB. Hence, the bolt diameter significantly affects the tensile behaviour of TubeBolt. Further experiments show that the bolt diameter does not enhance the tensile pull-out test capacity of the EHB due to the weakness of its anchorage system.

5.2.4 Cyclic performance of TubeBolt beam to CFT column connections

The experimental results indicate that the Novel TubeBolt beam connection serves as a rigid moment-resisting connection compared to an EHB beam connection.

The fabrication of a TubeBolt connection is practical and economical, as fewer connection components are utilized. The pre-welded extended end-plate on the beam end requires connecting elements to fix on the column face. For that reason, the bolts are passed through the pre-holed twin stiffener plates, extended end-plate and column and clamp the parts together. Elimination of the external stiffeners on the column face or internal stiffener inside the column, reduces the fabrication time.

The experimental results demonstrate that the anchor system of the TubeBolt is significantly capable of distributing the applied load into the CFT column.

5.2.5 Parameters affecting the TubeBolt connection performance

The experimental observations reveal that the TubeBolt significantly affects the failure mode of the connection. The connection with a TubeBolt and twin stiffener stiffener plates (specimens NBBC2 and NBBC4) exhibits a larger capacity and higher ductility compared to the EHB. Furthermore, column deformation, weld cracking, and flange fracture were not observed in the TubeBolt connection.

The experimental test of specimens NBBC2 and NBBC4 terminated due to the limited stroke of the actuators while the connections were able to proceed further story drift.

It is found that the twin stiffener plates also effectively affect the failure mode. The connections with twin stiffener plates (specimens NBBC2 and NBBC4) shows a much higher performance compared to the NBBC3.

The result of specimen NBBC6 with 10.9 bolt grade indicates that the connection is unable to be used in moment-resisting frames in seismic zones due to the significant bolt sliding, concrete crushing, column face deformation and economical specification.

The experimental study indicates that the connection properties, such as bolt type, bolt grade, twin stiffener stiffener plates, the extended end-plate thickness and bolt hole distance from beam flanges, have a significant effect on the failure modes, stiffness, strength, ductility, and energy dissipation of the proposed connection.

The twin stiffener plates shift the plastic hinge away from the extended end-plate face where fractures may occur due to the high rigidity, weld weakness, and stress concentrations in the weld access holes and bent column or flange. The results demonstrate that the cyclic performance, moment-resistance capacity, ductility, strength, stiffness, and energy dissipation of the novel TubeBolt beam connections significantly increased compared to the EHB.

5.2.6 Recommendations for future research

This thesis was proposed a novel blind bolt so-called TubeBolt for beam to CFT column connection. The pull-out test behaviour of the TubeBolt and cyclic performance of the TubeBolt connections were examined through the experimental programme. However, there are some areas which are required further studies. The suggestions for future research are proposed as below.

In this study, the pull-out and connection test used normal strength concrete with grades 40. Further studies are required to investigate the behaviour of the TubeBolt in concrete with various coarse aggregate size, lightweight concrete, high and low strength concrete mixes.

To evaluate the rotational behaviour of connections, a component method could be proposed. The model could be covered loading, unloading, ultimate strength, and sliding.

In this thesis, single bolt pull-out tests were performed. However, group bolts (with different gauge and pitch) pull-out test can be performed to study the effect of the overlapped cones on the pull-out test results.

Future studies can propose Finite Element modeling to simulate the pull-out performance of the TubeBolt and cyclic behaviour of the TubeBolt connection. The current data that was collected from the experimental test, could be used to validate the models. The Finite Element model can be utilized to perform parametric studies on the TubeBolt connection behaviour.

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