Seismic behaviour of engineered cementitious composites beam-column joints

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**NANYANG
TECHNOLOGICAL
UNIVERSITY**

**SEISMIC BEHAVIOUR OF ENGINEERED
CEMENTITIOUS COMPOSITES BEAM-COLUMN
JOINTS**

**LEE SIONG WEE
SCHOOL OF CIVIL AND ENVIRONMENTAL
ENGINEERING**

2017

**SEISMIC BEHAVIOUR OF ENGINEERED
CEMENTITIOUS COMPOSITES BEAM-COLUMN
JOINTS**

LEE SIONG WEE

School of Civil and Environmental Engineering

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SUMMARY

Reinforced concrete (RC) beam-column joints are one of the most important components that may lead to collapse or major damage of an RC frame structure when subject to earthquake loading. Structural engineers tend to pay attention to the detailing of the joints, which may lead to reinforcement congestion and thus making it difficult to be constructed. The emergence of engineered cementitious composites (ECC), which features ultra-ductility and damage-tolerance, offers a potential solution to the abovementioned problem. Therefore this study evaluates the feasibility of using ECC in the joint cores as a means to enhance the seismic behaviour of the RC beam-column sub-assemblages.

Two series of experimental tests were conducted on interior and exterior RC beam-column sub-assemblages under lateral cyclic loading. In the first series, two RC interior beam-column sub-assemblages were detailed to seismic design while another two were detailed to non-seismic design. The control specimens had normal concrete in the joint core while ECC was incorporated into the joint core of the seismic and non-seismic designed sub-assemblages. Joint stirrups were totally eliminated from the joint core when ECC was used in the beam-column sub-assemblages, to study the performance of ECC joints without joint stirrups. On the other hand, four exterior beam-column sub-assemblages were adequately designed to seismic in the second series of test. The effects of joint stirrups and column axial load on the seismic behaviour of the exterior beam-column sub-assemblages were investigated. Comparisons on the hysteretic response, failure mode, analysis of inter-story drift contribution, distribution of beam and column longitudinal bars strain, joint shear stress and energy dissipation were made between sub-assemblages with joint cores either made of ECC or RC materials.

In order to study the bond-slip behaviour of steel reinforcement embedded in ECC, a series of pull-out tests was conducted on short and long reinforcement. Thereafter an analytical model for bond-slip was proposed based on experimental results to predict the force-slip relationship of long reinforcement either anchored in concrete or in ECC. Moreover, a series of parametric study was conducted to estimate

required embedment length of reinforcement in concrete and ECC according to four types of failures. Finally, a beam-column joint model available in OpenSees was employed in simulating seismic response of the beam-column sub-assemblages. Bond strength of steel reinforcement embedded in ECC obtained from pull-out test was incorporated in the bar-slip component for ECC joints. Results of simulation were discussed.

LIST OF PUBLICATIONS

Lee, S.W., Kang, S.-B., Tan, K. H., and Yang, E.-H. (2016), *Experimental and Analytical Investigation on Bond-Slip Behaviour of Deformed Bars Embedded in Engineered Cementitious Composite*. Construction and Building Materials, 127, 494-503.

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LIST OF SYMBOLS

A_g	Gross area of column
A_b	Nominal steel bar area
b_c, b_w	Widths of column and beam, respectively
b_j	Effective width of joint
BI	Beam reinforcement index
d_b	Diameter of beam rebar
E_s	Modulus of elasticity of steel bars
E_h	Hardening modulus of steel bars
E_r	Softening modulus of steel bars beyond its ultimate strength
f_c'	Compressive strength of concrete
f_{cd}	Design compressive strength of concrete
f_s	Stress of steel reinforcement (bar stress)
f_y	Yield strength of steel reinforcement
h	Storey height of the column
h_b	Depth of beam
h_c	Depth of column
jd	Distance between the internal tension and compression force resultants in the beams
JI	Joint transverse reinforcement index

k	confinement coefficient for concrete model
L	Embedment length of steel bars
l	Horizontal distance between the beam supports
l'_{bi}, l'_{ci}	Distance measured from section i to the beam and column end support, respectively
l_e, l_y	Length of the steel bar for which bar stress is less than or greater than the yield strength, respectively
M_{Rb}, M_{Rc}	Design flexural moments from beams and columns, respectively.
ρ	Ratio of the volume of transverse reinforcement to the volume of concrete core measured to the outside of stirrups
s	Slip of steel reinforcement embedded in concrete
V_j	Horizontal joint shear force
V_c	Column shear force
v_{jh}	Nominal horizontal joint shear stress
v_j	Joint shear stress
v_d	Normalised column axial
ϕ_0	Overstrength factor for beams
ϕ	Diameter of steel reinforcement
Δ_{bf}, Δ_{cf}	Displacement due to flexural deformations of beams and columns, respectively
Δ_j	Displacement due to shear distortion of the joint
δ_{bf}, δ_{cf}	Flexural deformations of beams and column, respectively
$\delta k_i, \delta f_i, \delta d_i$	Damage index for stiffness, strength and reloading strength for

shear panel.

$\theta_{b,i}, \theta_{c,i}$	Rotation of beams and column at section I, respectively
γ_j	Joint shear distortion
ε_y	Yield strain of steel reinforcement
ε_u	Ultimate strain of steel reinforcement
ε_r	Fracture strain of steel reinforcement
ε_o	Strain of unconfined concrete corresponding to f'_c
τ	Bond stress of steel reinforcement embedded in concrete
τ_{ET}	Average elastic bond strength for a bar in tension
τ_{EC}	Average elastic bond strength for a bar in compression
τ_{YT}	Average inelastic bond strength for a bar in tension
τ_{YC}	Average inelastic bond strength for a bar in compression

CHAPTER 1: INTRODUCTION

1.1 Research Background

When subject to seismic actions, reinforced concrete (RC) beam-column joints are one of the most important components that may lead to collapse or major damage of RC frame structures. In this regard, seismic design codes pay specific attention on the detailing of joint cores to prevent brittle joint failures. For instance, three main aspects in RC beam-column joint design are emphasised by the Joint ACI-ASCE (ACI-ASCE 2002) design code, viz. confinement reinforcement requirement, evaluation of shear strength, and anchorage of beam and column bars passing through joints. Nevertheless, seismic design of RC beam-column joints may lead to reinforcement congestion in the joint regions, and cause construction difficulties on site.

To avoid the aforementioned problem, a ductile concrete material which requires no transverse reinforcement could be used. In the past two decades, advances in modern concrete technology enable the development of engineered cementitious composites (ECC), a unique high performance fiber-reinforced cementitious composites which features ultra-high ductility and damage tolerance under tensile and shear loading (Li 2003). **Fig. 1.1** shows a qualitative comparison between typical tensile stress-strain curves of ECC, fiber-reinforced concrete (FRC) and conventional concretes. It is observed that ECC exhibits tensile strain-hardening behaviour beyond the first cracking and its strain capacity is substantially higher than that of conventional concrete and FRC. Additionally, ECC shows multiple-cracking behaviour at small crack spacing (0.5 to 5 mm) and small individual crack widths ($< 200 \mu\text{m}$) (Fischer and Li 2003).

The pseudo strain-hardening tensile properties of ECC materials can be successfully translated into enhanced shear performance over conventional concrete and FRC, as demonstrated by Li et al. (1994). Therefore, it is suggested that ECC can be exploited in applications where superior ductile performance is desired. In compression, ECC reaches its compressive strength at larger strain due to the lack

of aggregates (Fischer and Li 2003), and its ultimate compressive strain is generally larger than in normal concrete. Hence, it can relax confinement links requirements in beam-column joints.

Fig. 1.1: Schematic stress-strain behaviour of cementitious matrices in tension (Fischer and Li 2003)

Further more, compatible deformations between steel reinforcement and ECC (Fischer and Li 2002; Kang et al. 2015; Li and Fischer 2002) can reduce required embedment length of rebars passing through beam-column joints. The advantages of ECC over conventional concrete make it an ideal material for beam-column joints that can resist high shear stress under cyclic actions. Therefore, it is important to investigate the feasibility of ECC in achieving large displacement capability and high damage tolerance of beam-column joints which required no or minor post-earthquake repairs.

Previous experimental investigations have revealed the advantageous behaviour of ECC in beam-column joints subject to cyclic loading. When ECC was used in beam-column joints without any links, the joints exhibited superior damage tolerance and sustained substantial shear distortions under cyclic loads (Parra-Montesinos 2005; Parra-Montesinos et al. 2005; Said and Razak 2016; Zhang et al. 2015). In the presence of links, ECC joints significantly increased the load capacity,

ductility and energy dissipation of the beam-column sub-assemblages (Yuan et al. 2013).

Even though the performance of ECC was experimentally investigated as abovementioned (Parra-Montesinos and Wight 2000; Parra-Montesinos et al. 2005; Qudah and Maalej 2014; Said and Razak 2016; Yuan et al. 2013; Zhang et al. 2015), there is still some missing information. For instance, in some of the studies (Parra-Montesinos et al. 2005), performance of beam-column joints with ECC joints could not be compared with normal concrete specimens as there was no control specimen in the test programme. In addition, some studies did not address the joint shear strength (Said and Razak 2016; Yuan et al. 2013) of beam-column joints with ECC. This made assessment of shear strength of ECC joints challenging, and test results could not be compared with code-defined limiting shear strength values. To address the aforementioned limitations in the existing literature, experimental investigations have to be carried out to gain a deeper knowledge on the behaviour of beam-column joints made of ECC when subject to cyclic loading.

It is particularly important to evaluate the effect of ECC in enhancing bond-slip behaviour of beam longitudinal bars passing through the joints compared with concrete. Limited information in this regard has been reported in previous beam-column joint tests (Parra-Montesinos et al. 2005; Yuan et al. 2013; Zhang et al. 2015). It is difficult to quantify bond stress through experimental tests on beam-column joints (Kang 2015). Therefore, pull-out tests have to be carried out to study the bond-slip behaviour of steel reinforcement embedded in ECC cores.

Furthermore, limited attention has been paid on the mechanical modelling of the joints made of ECC. A simpler approach called “component-based joint model” has been proposed to idealise the force transfer at the perimeter of a joint as a series of simple uni-axial components to explicitly represent the inelastic joint behaviour. There are many proposed beam-column joint models under seismic action. Among them, the beam-column joint models proposed by Lowes et al. (2003) enables simulation of inelastic joint response due to anchorage bond failure (bar-slip springs), joint shear failure (shear panel) and interface-shear transfer (interface-shear springs). The constitutive relationships of each component are defined as a

function of design parameters, material and geometry properties. Since ECC material is used in the joint core, it is necessary to evaluate the material models for each component in the beam-column joint model proposed by Lowes et al. (2003).

1.2 Objectives and Scope of Research

This research aims to evaluate the feasibility of using ECC in RC beam-column sub-assemblages subject to cyclic loading through experimental tests. Previous research incorporated ECC material in the joint cores and throughout the plastic hinge length of beams and columns (Parra-Montesinos et al. 2005; Qudah and Maalej 2014; Said and Razak 2016; Yuan et al. 2013; Zhang et al. 2015). However, in this study, ECC material was only incorporated in the joint cores for beam-column sub-assemblages. This is because ECC material is approximately four times more expensive than normal concrete and its use should be kept to the minimum. The primary objectives of this research are as follows:

- (1) To investigate the seismic behaviour of interior beam-column sub-assemblages with ECC in the joint cores. In this experimental programme, RC beam-column sub-assemblages are designed under both non-seismic and seismic scenarios according to Eurocode 2 (BSI 2004) and Eurocode 8 (BSI 2004), respectively. The performance of ECC joints in sub-assemblages is evaluated for both seismic and non-seismic design scenarios. Their behaviours are compared to specimens made of normal concrete joints.
- (2) To investigate the seismic behaviour of exterior beam-column sub-assemblages with ECC in the joint cores. All exterior beam-column sub-assemblages were seismically designed and tested under lateral cyclic load at the top of column. A comparison study on the seismic behaviours of ECC specimens and concrete specimens is made. The effect of joint stirrups and column axial load on the seismic behaviour of sub-assemblages with ECC in the joint cores is studied.
- (3) To study the bond-slip behaviours of steel reinforcement embedded in ECC through pull-out tests. This test involves both short and long embedment lengths of steel reinforcement in well-confined concrete and ECC blocks. The maximum bond stress by ECC during elastic and post-yield stages is quantified.

(4) To develop an analytical model for predicting force-slip relationship for long embedment of reinforcement in ECC based on pull-out test result. A parametric study is conducted to quantify the required embedment length of steel reinforcement in ECC.

(5) To simulate seismic response of the beam-column sub-assemblages using component-based joint model. Special attention is placed on the bar-slip behaviour, in which the enhancement of bond by ECC is incorporated in the bar-slip material model.

1.3 Layout of Thesis

This thesis consists of seven chapters. The contents are briefly described as follows:

Chapter 1 describes the background and gaps of research. Objectives and scopes of current research are defined.

Chapter 2 presents an overview on seismic design approaches for RC beam-column joints. Previous experimental tests on RC beam-column joints are reviewed. ECC materials and its structural applications are introduced, followed by previous tests on beam-column joints with ECC. Moreover, component-based joint model is briefly introduced.

Chapter 3 describes the design of specimens for interior and exterior beam-column sub-assemblages. The test set-up, together with instrumentation and loading history are included in this chapter.

In Chapter 4, test results for both interior and exterior beam-column sub-assemblages are presented. Materials properties for concrete, steel reinforcement and ECC are reported. The performance of beam-column sub-assemblages is evaluated through a comparison study of hysteresis response and failure mode, analysis of inter-story drift contributions, joint shear strain, strain reading and energy dissipation with conventional concrete specimens. Conclusions are drawn based on experimental results of each series of test.

Chapter 5 focuses on the bond-slip behaviour of steel reinforcement embedded in ECC. Pull-out tests are carried out experimentally and bond stress of steel reinforcement embedded in concrete and ECC is quantified. Besides, an analytical

model is proposed to predict force-slip relationships for long embedment of reinforcement. A series of parametric studies is then conducted to determine the require embedment length of reinforcement in concrete and ECC based on the different types of failure.

In Chapter 6, numerical simulation on seismic response of beam-column sub-assemblages is conducted. Material models for each component in the beam-column joint model are described in detail. Input parameters for using the software OpenSees to generate the predictions are included. Results of simulations are discussed and justification is given on the choice of material models for ECC joints.

Chapter 7 summarises the conclusions of the entire research works. Future works are also addressed.

An Appendix is enclosed in this thesis on design of test specimens.

CHAPTER 2: LITERATURE REVIEW

2.1 Overview

This chapter presents an overview on seismic design approaches for reinforced concrete frames, mainly on the design of beam-column joints. Previous experimental tests on both interior and exterior beam-column joints under cyclic loading are reviewed to shed light on the brittle failure of the joints. Besides, ECC and its performance in seismic-resistant structures are discussed with supporting evidences from previous tests. Lastly, component-based joint model is introduced for simulating seismic behavior of beam-column joints.

2.2 Seismic Design of Beam-column Joints

2.2.1 Strong-column-weak-beam approach

When a moment-resisting frame is seismically loaded, yielding should begin in the beams rather than in the columns as this is the preferred mechanism since plastic hinges can be formed in the beams and localised damage in column can be reduced. This can be achieved through designing relatively strong columns compared to the beams. **Fig. 2.1** shows the mode of behaviour for a moment-resisting frame in the post-elastic range under seismic loading. If yielding begins in the columns before it occurs in the beams, a column sidesway mechanism is formed as shown in **Fig. 2.1(b)**. In this case, drift tends to concentrate in one or a few storey and may exceed the drift capacity of columns (Moehle 2008). This mechanism is undesirable for a building over two-storey height since it can place very large demands on ductility at plastic hinge locations in the columns of the critical storey (Park and Paulay 1975). Most of the seismic design codes specify that columns must be stronger than beams in a moment-resisting frame to avoid column sidesway mechanism, i.e. a strong-column-weak-beam approach is advocated. To ensure that columns are stronger than beams in moment resisting frames, seismic design codes require the column moment resistance should exceed that of the beams at the beam-column joints. **Table 2.1** gives a comparison of capacity design ratios for column-to-beam flexural strengths from three design codes, viz. Eurocode 8 (BSI 2004), joint ACI-ASCE 352 (ACI-ASCE 2002) and New Zealand Standard (Standard 2006). In the **Table**

2.1, $\sum M_{Rc}$ and $\sum M_{Rb}$ are the sum of design moments from columns and beams converging to the joint respectively; ϕ_0 is the overstrength factor for beams, which may be taken as 1.47 (Uma and Jain 2006).

Fig. 2. 1: Modes of behaviour for tall seismically loaded moment resisting frames in the post-elastic range (Park 2002)

Table 2.1: Comparison of capacity design check

Codes	Capacity check
EC 8	$\sum M_{Rc} \geq 1.3 \sum M_{Rb}$
Joint ACI-ASCE	$\sum M_{Rc} \geq 1.2 \sum M_{Rb}$
NZS 3101	$\sum M_{Rc} \geq 1.4 \sum \phi_0 M_{Rb}$

Notable the capacity check in NZS as shown in **Table 2.1** was the simplified version provided by Uma and Jain (2006), in which dynamic magnification had been accounted for the ease of comparison. Among the three codes, NZS requires a higher value of column flexural strength over beam flexural strength. The design flexural strength of adjoining columns is expected to be at least 2.06 times higher than the design flexural strength of adjoining beams (Uma and Jain 2006) if $\phi_0 = 1.47$ is considered.

2.2.2 Behaviours of beam-column joints

The concept of strong-column-weak-beam is to prevent the overall building collapse and minimise local damage under severe seismic excitations. In order to maintain good integrity between beam and column elements, joints are recognised to be the most essential part of a structure to transfer forces from adjacent beams and columns during seismic loading. **Fig. 2.2(a)** shows the resultant forces from beams and columns that act on joint cores; joint shear forces are developed through horizontal and vertical faces at the four corners of joints. The joint shear forces are in the form of diagonal tension and diagonal compression as shown in **Fig. 2.2(a)**. In New Zealand, two shear-transferring design approached method are recommended. The first method is truss mechanism (**Fig. 2.2(b)**), in which shear is mainly resisted by shear reinforcement (both horizontal and vertical). This shear reinforcement will mobilise concrete diagonal compression field (Paulay and Park 1984), and hence the effective truss mechanism can prevent shear failure by diagonal tension. Shear forces can be transferred via diagonal strut mechanism in the joint as shown in **Fig. 2.2(c)**, in which concrete is resisting shear force predominantly by its compressive strength. In order to prevent concrete diagonal strut from crushing, an upper limit of horizontal stress acting on the joint is defined in most of the design codes. To evaluate the horizontal shear stress acting on the joint, design codes have specified their assumptions for effective joint area. Basically, joint shear stress is not critical for exterior joints since horizontal shear forces transferred from the beams is only half as much as that in interior joints. Therefore the limiting joint shear stresses for exterior joints can be relaxed for around 20%. **Table 2.2** shows the allowable nominal joint shear stress and the definition of effective joint width given by the three codes. EC8 considers the effect of column axial load in determining nominal joint shear stress while ACI-ASCE evaluates nominal joint shear stress based on confinement provided by the adjoining members. For effective joint area, EC8 has similar manner of determining the effective width as NZS; the effective width depends on the column width which may be greater or smaller than the beam width.

Crushing of joint core is unfavourable because it will cause a dramatic loss of strength, stiffness and ductility. Therefore beam-column joints must be sufficiently

ductile so that failure would occur in beams which are designed according to strong-column-weak-beam concept. Flexural mode of failure in beams yields a more stable load-displacement hysteresis loop, associated with significantly less degradation of strength, stiffness and energy dissipation.

Fig. 2. 2: Interior beam-column joint subject to seismic loading (Hakuto et al. 2000)

Anchorage failure is categorised as one of the brittle failure modes in beam-column joints. When the bond strength of rebars passing through the joint is lost, rebars may be pulled out and total slip results from the beam-column interfaces. Anchorage failure gives rise to significant inelastic deformation of the beam-

column joints and leads to collapse or local damage of beam-column sub-assemblages.

Table 2.2: Nominal joint shear stress and effective width of joint

Codes	Nominal joint shear stress	Effective width of joint, b_j
EC 8	<p>For interior joints,</p> $v_{jh} = \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}}$ $\eta = 0.6 \left(1 - \frac{f_c'}{250}\right)$ <p>where v_{jh} is the nominal joint shear stress; f_{cd} is the design compressive strength; f_c' is the compressive strength of concrete; v_d is the normalised axial in the joint above column.</p> <p>For exterior joints, v_{jh} is taken as 80% of calculated as in interior joints.</p>	<p>if $b_c > b_w$, $b_j = \min \{b_c; (b_w + 0.5h_c)\}$</p> <p>if $b_c < b_w$, $b_j = \min \{b_w; (b_w + 0.5h_c)\}$</p> <p>where b_c and b_w are the widths of the column and the beam respectively; h_c is the column depth.</p>
Joint ACI-ASCE	$v_{jh} = 0.083\gamma\sqrt{f_c'}$ <p>Where</p> <p>$\gamma = 15$ for joints effectively confined on all four vertical faces.</p> <p>$\gamma = 12$ for joints effectively confined on three or two opposite vertical faces.</p> <p>$\gamma = 10$ for other cases.</p>	$b_j = \min \left\{ \frac{b_w + b_c}{2}; b_w + \sum \frac{mh_c}{2}; b_c \right\}$ <p>where m is taken as 0.3 when the eccentricity between the beam centreline and the column centroid exceeds $b_c/8$; or else m is taken as 0.5.</p>
NZS 3101	$v_{jh} = 0.2f_c' \text{ or } 10 \text{ MPa}$	<p>if $b_c > b_w$, $b_j = \min \{b_c; (b_w + 0.5h_c)\}$</p> <p>if $b_c < b_w$, $b_j = \min \{b_w; (b_c + 0.5h_c)\}$</p>

Fig. 2.3 shows the bar stress and bond stress distribution along a single rebar passing through an interior beam-column joint. Upon the onset of first yielding in the rebar, a uniform bar stress variation is assumed which results in a constant bond stress, u_1 is formed as shown in **Fig. 2.3(a)**. After reversal of inelastic loading,

compressive stress reaches yield and the resultant bond stress, u_2 , can be slightly greater than u_1 . However, after several cycles of reversed inelastic loading, yielding penetration inevitably occurs along the beam bar into the joint core when the inelastic bar strains rapidly increase (Paulay and Park 1984). Bond stress is greatly increased to u_3 . But bond compatibility is destroyed and the anchorage length is reduced as shown in **Fig. 2.3(c)**. As a result, partially bars may slip through the joint core and cause a great loss of stiffness and energy dissipation for the entire structural system. Moreover, slippage of rebars lead to a loss of compressive forces in the reinforcement as the rebar switches from compression to tension. Thus, flexural strength and curvature ductility is reduced subsequently (Hakuto et al. 1999). Therefore, seismic design codes have provisions to specify the maximum ratio of the size of beam rebars, d_b , to the column depth, h_c , where the rebar passing through. For instance, ACI-ASCE specifies $\frac{h_c}{d_b} \geq 20 \frac{f_y}{420} \geq 20$ MPa. This limit is to control the slip so that bond anchorage failure can be avoided. Therefore, in general case, a low level of bar-slip is acceptable as long as ductility demand is achieved.

Fig. 2. 3: Bar stress and bond stress distribution along bar passing through an interior beam-column joint (Paulay and Park 1984)

2.3 Tests on RC Beam-Column Joints

Due to the importance of beam-column joints in the response of concrete frames under seismic loading, comprehensive research have been conducted since a few

decades ago. For non-seismic detailing joints, the absence of transverse reinforcement in joints causes diagonal tensile failure before crushing of diagonal strut can take place as demonstrated by previous tests (Hakuto et al. 2000). In Hakuto's test, one of the interior beam-column joints was designed according to NZS 3101 for non-seismic detailing and tested under lateral cyclic loads. The ratio of beam bar diameter to column depth was $d_b/h_c = 24/300 = 1/12.5$ which did not satisfy the requirements of most of the codes. **Fig. 2.4** shows the tested specimen, load-displacement response and cracks patterns. Failure of this specimen was due to diagonal tensile cracking and bond-slip in the joint, thus pinching and severe degradation of stiffness can be observed in the hysteresis loops (**Fig. 2.4(b)**). The maximum nominal shear stress was calculated as $0.095f'_c$, equivalent to $0.61\sqrt{f'_c}$ in which crushing did not occur. It is noteworthy that the concept of strong-column-weak-beam was not applied, and the ratio of column flexural strength to beam flexural strength was merely 0.69. Plastic hinges were expected to form in the columns, but flexural strength was not achieved in the negative loading direction due to shear failure occurred in the joint core. Notably, this test demonstrated the importance of adequate anchorage length of beam bars passing through the joint cores.

(a) Test specimen (all units are in mm)

(b) Load-displacement response

(c) Cracks pattern

Fig. 2.4: Test specimen, load-displacement response and cracks pattern (Hakuto et al. 2000)

Ehsani and Wight (1985) investigated the effect of three variables, viz. the ratio of the column to beam flexural capacity (flexural strength ratio), joint shear stress, and transverse reinforcement in the joint on seismic performance of six exterior beam-column joints. **Table 2.3** gives the parameters for the six specimens while **Fig. 2.5**

shows the corresponding load-displacement responses. For specimens with flexural strength ratio slightly greater than 1.0, flexural hinges formed in the beams but spread into the joints. This led to pull-out of longitudinal beam bars and caused the severe degradation of strength and stiffness as shown by specimens 1B and 3B. Compared with limiting shear stress as specified by ACI-ASCE code, i.e. $1.0 \sqrt{f_c'}$ MPa for exterior joints, most of the specimens were closed or exceeded this limit. Specimen 6B with a shear stress of only $0.67 \sqrt{f_c'}$ MPa remained its maximum load capacity throughout a few cycles and demonstrated good ductility (**Fig. 2.5(f)**). This specimen showed that in order to obtain ductile behaviour of joints, shear failure should be avoided. When the amount of transverse reinforcement increased in the joints (1.30% and 1.48% respectively for specimens 3B and 4B), maximum load capacity increased accordingly. However, the effect of flexural strength ratio was significant in promising ductility of beam-column joints as shown in **Figs. 2.5(c and d)**. This study revealed that the amount of transverse reinforcement could be reduced when the flexural strength ratio is greater than 1.4 and the joint shear stress is kept below $1.0 \sqrt{f_c'}$ MPa as demonstrated by specimen 6B.

Table 2.3: Variables of exterior beam-column joints (Ehsani and Wight 1985)

Specimen	Flexural strength ratio	Joint shear stress (MPa)	Joint hoops percentage (%)
1B	1.01	$1.10 \sqrt{f_c'}$	0.87
2B	1.35	$1.10 \sqrt{f_c'}$	0.98
3B	1.07	$1.00 \sqrt{f_c'}$	1.30
4B	1.41	$1.01 \sqrt{f_c'}$	1.48
5B	1.93	$1.18 \sqrt{f_c'}$	0.78
6B	1.56	$0.67 \sqrt{f_c'}$	0.74

2.4 Engineered Cementitious Composites (ECC)

In the family of fibre-reinforced concretes (FRC), the behaviour of high-performance fibre-reinforced cementitious composites (HPFRCC) is characterised by strain-hardening behaviour under direct tension. This strain-hardening behaviour implies high ductility, toughness and large energy absorption capacity, which distinguishes HPFRCC from conventional FRC (Li et al. 2001). Large volume fraction of fibres (typically above 5%) has to be used in HPFRCC such as slurry infiltrated fibre concrete (SIFCON) and slurry infiltrated mat concrete (SIMCON) to achieve strain-hardening behaviour (Li et al. 2001; Parra-Montesinos 2005), but engineered cementitious composites (ECC) differs from the others by adopting short fibres at typically moderate volume fraction (1 to 2%). Its ultimate tensile strain capacity can be up to 5% while its crack width is limited to below $100\mu\text{m}$ (Li et al. 2001). The key to achieve such superior tensile behaviour in ECC is the application of micromechanics tool, which allows microstructure tailoring and optimization (Li et al. 2001; Yang and Li 2010). Microstructure tailoring consists of deliberate selection or modification of fibre, matrix, and interface properties, including the addition of nano-size particles and fibre surface coating. Therefore, it is the constituent tailoring and optimization methodology embodied that accounts for its names as Engineered Cementitious Composite or ECC in short (Li 2003).

(a) 1B

(b) 2B

Fig. 2.5: Load-displacement response (Ehsani and Wight 1985)

2.4.1 ECC as an alternative for seismic design of structures

ECC generally exhibits tensile strain capacities between 1.0-5.0%, depending on the type of fibres used, matrix composition and matrix-fibre interface. **Fig. 2.6(a)** shows an example of tensile stress-strain relationship of ECC and conventional concrete. It can be seen clearly that normal concrete fails in a brittle manner once reaching its cracking strength, while ECC is capable of undergoing a strain-hardening phase. Beyond first cracking, ECC exhibits strain-hardening behaviour with the development of multi-cracking, up to a strain level of several percents. This superior behaviour makes it attractive to be used in earthquake-resistant

structures. Under compression (**Fig. 2.6(b)**), ECC has a lower elastic modulus compared with normal concrete due to the lack of coarse aggregates, but it reaches its compressive strength at a larger strain (Fischer and Li 2003). Its greater compressive strain also contributes in enhancing the seismic performance of structures. The post-peak response reveals that ECC is well confined, and therefore, it relaxes requirements for confinement reinforcement in critical regions of earthquake-resistant structures such as the beam-column joints. Therefore, both tensile and compressive behaviour make ECC an ideal material in seismic-resistant structures. Nevertheless, study by Yuan et al. (2013) showed that ECC was more effective in resisting flexural loads when applied in the tension zone rather than the compression zone of a beam.

Fig. 2.6: Stress-strain behavior of ECC and concrete (Fischer and Li 2003)

2.4.2 Performance of ECC structural members

The use of ECC in structural members is encouraged by JSCE design guideline (JSCE 2008). In comparison with normal concrete and FRC, ECC possesses a higher load-carrying capacity, ductility, durability and energy absorption capacity. Therefore, excellent seismic performance can be observed in ECC structural members subject to large inelastic deformations, or high shear stress reversals. Previous experimental investigations have revealed the advantages of ECC in structural applications. These beneficial characteristics of ECC structural members are outlined as follows.

2.4.2.1 Shear resistance

Li et al. (1994) demonstrated that pseudo-strain-hardening properties of ECC can be successfully translated into advantageous structural shear response through a series of Ohno shear beam tests. Thereafter, similar tests were conducted by Shimizu et al. (2004) and Fukuyama et al. (2000). A pure shear test at stress-strain level conducted by van Zijl (2007) showed that ECC exhibited high shear resistance and ductility. At structural level, it has been concluded by several researchers that the ductile nature of ECC could be used effectively to improve the performance of shear-dominated structural members. Members subjected to high shear generally require extensive transverse reinforcement, and yet previous research has shown that substantial reduction or even elimination of transverse reinforcement can be achieved through the use of ECC (Li 2003; Parra-Montesinos 2005). It is noteworthy that flexural member tests by Fischer and Li (2002) confirmed that specimen R/ECC (reinforced ECC) without stirrups performed better than specimen R/C (reinforced concrete) with stirrups. **Fig. 2.7** shows the corresponding damage pattern of the two specimens. No spalling was observed in specimen R/ECC whereas loss of concrete cover after bond splitting and spalling has occurred in specimen R/C. This reveals that ECC provides sufficient shear resistance for R/ECC member, and it also served as lateral confinement for the longitudinal reinforcing bars in the column. In the test by (Li and Wang 2002), ECC beams without shear reinforcement exhibited better flexural stiffness, ductility and load-carrying capacity than the high strength concrete (HSC) beam with dense shear reinforcement. This remarkable improvement in shear resistance reflects that eliminating shear reinforcement is feasible when ECC is used.

(a) R/C

(b) R/ECC

Fig. 2.7: Damage behaviour of the tested flexural member (Fischer and Li 2002)

2.4.2.2 Energy absorption capacity

Billington and Yoon (2004) showed that the use of ductile fiber reinforced cementitious composites (DFRCC) at potential plastic hinge regions of precast segmental concrete piers allowed the system to dissipate 50% more hysteretic energy than conventional concretes up to a drift level of 3-6%, while maintaining its integrity under high cyclic tensile-compressive load reversals. Similar observations were obtained from tests by Fischer and Li (2002), where energy absorption of R/ECC member was almost three times greater than that of R/C member (**Fig. 2.8**). In both tests, even though no transverse reinforcement was provided to the specimens with ECC, the hysteretic responses were stable. It is believed that the ductile stress-strain behaviour of ECC allows high deformations in reinforcement and leads to better energy dissipation capacity.

Fig. 2.8: Hysteresis loops of column members under fully cyclic loading (Fischer and Li 2002)

2.4.2.3 Damage tolerance

Damage tolerance is a measure of residual strength of a material or a structure when damage is introduced and it can be reflected in the failure mode (Li 2003). The high damage tolerance of ECC makes it ideal for use in structural elements where high stress concentration is unavoidable such as beam-column joints. Beam-column joints incorporating ECC have been tested by Parra-Montesinos and Wight (2000) and Parra-Montesinos et al. (2005). Parra-Montesinos and Wight (2000) proposed ECC to be used in hybrid reinforced concrete column-steel beam (RCS) connections, to evaluate the potential of ECC as replacement of joint transverse reinforcement. No fracture failure of longitudinal rebars was observed. However, severe spalling and fracture failure were observed in the R/C column. Similarly, in another beam-column connection test by Parra-Montesinos et al. (2005), minor damage was observed at the end of testing, despite no transverse reinforcement was provided in the beam plastic hinge region.

2.4.2.4 Compatible deformation between ECC and reinforcement

Another unique advantage of ECC is that in ECC structures (with reinforcement), both steel reinforcement and ECC remain compatible in deformation even if steel reinforcement yields. This is an important aspect to prevent local damage induced by bond slip between steel bars and ECC. Unlike normal concrete, ECC did not break apart due to excellent fibre bridging strength. Since stress can be transmitted

directly through ECC even after micro-cracking occurred, bond splitting and spalling of concrete can be avoided when ECC is used as shown in **Fig. 2.9** (Fukuyama et al. 2000).

(a) Bond Splitting Failure in RC beam

(b) Flexural Yielding in PVA-ECC beam

Fig. 2.9: Failure modes of specimens after Testing (Fukuyama et al. 2000)

Tension-stiffening behaviour of steel-reinforced ECC was studied through uniaxial tension tests by Fischer and Li (2002) and Moreno et al. (2014). **Fig. 2.10** shows the load-deformations of R/C composites, R/ECC composites and bare steel reinforcement under uniaxial tension. Prior to the first cracking, both specimens behaved in a similar stiffness. After the first cracking, transverse cracking caused a reduction of stiffness for R/C composites. The differences between R/C composites or R/ECC composites and bare reinforcement show the tension-stiffening effect. After cracking, tension stiffening effect in R/ECC composites was more significant compared to R/C composites due to compatible deformation of reinforcement and ECC matrix. Since ECC matrix can transfer stress across cracks, multiple cracking behaviour of ECC allows the reinforcement and the ECC matrix to deform together.

Fig. 2.10: Load-deformation response under uniaxial tension (Fischer and Li 2002)

2.4.3 Bond stress of reinforcement in ECC

To investigate the bond behaviour of steel reinforcement embedded in ECC, pull-out bond tests (Asano and Kanakubo 2012; Kanakubo and Hosoya 2015) and beam bond test (Bandelt and Billington 2014) have been carried out. Bandelt and Billington (2014) investigated bond-slip behaviour of steel reinforcement embedded in four composites, namely, concrete, ECC, self-consolidating high performance fibre reinforced concrete (SC-HPFRC) and self-consolidating hybrid fibre reinforced concrete (SC-HyFRC). The effect of concrete cover (ratio of concrete cover to diameter of beam bar) and confinement by stirrups on bond strength were studied. All specimens failed in bond splitting due to insufficient concrete cover. Results showed that when the ratio of concrete cover to diameter of beam bar is 1, and with no stirrups in the beams, reinforcement embedded in ECC exhibited the greatest bond compared with the others as indicated in **Fig. 2.11(a)**. But there was little effect on the bond stress when confinement was added in the ECC beam. However, bond strength of reinforcement embedded in concrete, SC-HPFRC and SC-HyFRC was improved with confinement (**Fig. 2.11(b)**). It implies that ECC can offer better confinement effect when stirrups are not provided.

Fig. 2.11: Bond stress-reinforcement slip response (Bandelt and Billington 2014)

Studies described above were limited to bond strength of ECC in terms of bond-splitting failure. However, with sufficient concrete cover and confinement reinforcement, pull-out failure instead of splitting failure occurred for short embedment of steel reinforcement in concrete blocks. A bond-slip model for pull-out failure of steel reinforcement embedded in concrete is well established in *fib* model code for concrete structures (Code 2010) as well as in the study by Eligehausen et al. (1983). Since the anchorage bond of beam bars passing through joints is closer to this condition, investigation of bond-slip behaviour is needed for steel reinforcement embedded in ECC.

2.4.4 Previous tests on beam-column joints incorporating ECC

Seismic performance of ECC in beam-column joints has been evaluated by previous researchers as listed in **Table 2.4**. They have demonstrated the possibility of using ECC as a total replacement of confinement reinforcement in the beam-column joints when subject to cyclic loading. **Table 2.4** also indicates the different material properties of ECC employed in previous tests. Tensile strain capacities for ECC ranging from 1.0 to 4.0% have improved seismic performance of beam-column joints. More details on the tests are described and the gaps of research are highlighted in the following section.

Table 2.4: Previous tests on beam-column joints incorporating ECC

Researcher (year)	Exterior/ Interior beam- column joint	Region incorporated by ECC	Fibres usage	Material properties of ECC		
				Tensile strain capacity (%)	Tensile strength (MPa)	Compressive strength (MPa)
Parra-Montesinos et al. (2005)	interior	Joint core and adjacent beam plastic regions	1.5% volume fraction of polyethylene (PE) fibres	SP1 1.0 SP2 1.3	2.7 2.2	39.3 (43.9*) 42.7 (41.3*) *For concrete
Yuan et al. (2013)	exterior	Joint core and adjacent beam and column plastic regions	Polyvinyl alcohol (PVA) fibres	4.0	5.0	49.6 (52.4*) * For concrete
Qudah and Maalej (2014)	interior	Joint core and adjacent beam and column plastic regions	2.0% volume fraction of polyethylene (PE) fibres	Not given	Not given	Varied from 43.0-52.0
Zhang et al. (2015)	exterior	Joint core and the whole span of beam	3.0% volume fraction of polypropylene (PP) fibres	3.0	3.5	TJ-1 48.2(50.0*) TJ-2 33.6(45.2*) TJ-3 33.6 * For concrete
Said and Razak (2016)	exterior	Joint core and adjacent beam and column plastic regions	2.0% volume fraction of polyvinyl alcohol (PVA) fibres	1.0	4.0	Not given

2.4.4.1 Parra-Montesinos's test

Two approximately $\frac{3}{4}$ -scaled beam-column sub-assemblages, which represented typical connections where beams were framed into the column from two opposite sides, were tested under displacement reversals. Both specimens were designed according to strong-column-weak-beam approach. The nominal column moment strength to beam strength ratios for Specimens 1 and 2 were 2.2 and 1.6, respectively. **Fig. 2.12** shows the frame configurations of the two specimens. ECC material was incorporated in the beam-column joint and adjacent beam region (i.e. twice the beam depth). All the shear reinforcement was eliminated in the beam-column joint. Special transverse reinforcement was provided in the column region just above and below the beams to spread out the beam inelastic deformations. The same transverse reinforcement was provided throughout the beam span. Two layers

of intermediate reinforcements in the connection region were used to enhance joint post-cracking behaviour. A displacement-control reversed cyclic load was applied, ranging from 0.5% to 5.0% drift ratio. Two cycles were performed in each drift ratio. Besides, a constant low axial load (i.e. 4% of column axial capacity) was applied through hydraulic jacks. It is noteworthy that the ratio between the column depth and the diameter of beam longitudinal reinforcement was equal to 18.7, which is slightly below the minimum ratio of 20 as specified in the ACI Building Code. Experimental results demonstrated that both specimens performed satisfactorily, with excellent strength, deformation capacity and damage tolerance, even though no special confinement reinforcement was provided. The peak shear stress demands ($1.2\sqrt{f_c'}$ and $1.4\sqrt{f_c'}$) sustained by the two specimens indicated that ACI shear stress limits of ($1.25\sqrt{f_c'}$) are adequate for use in ECC joints with no confinement reinforcement. Brittle joint shear failure was not observed in any of the tested specimens; only minor damage and multiple cracks appeared at the joint region. The peak joint shear distortion was 0.002 rad with maximum crack width of 0.6 mm for specimen 1 and 0.008 rad with maximum crack width of 3.0 mm for specimen 2, respectively, indicating a good damage-tolerant behaviour of ECC in the joint.

Also, no bond deterioration in the beam longitudinal rebars passing through the beam-column connection was observed in this test. **Fig. 2.13** shows the bond efficiency of the top and the bottom beam bars versus drift of the two specimens. Bond efficiency values greater than 1 were attained during drift of 3-6%, indicating good bond behaviour of ECC even though the requirement of anchorage bond length was not fulfilled.

Fig. 2.12: Frame Configuration and Reinforcing Details (all units are in mm)
(Parra-Montesinos et al. 2005)

Fig. 2.13: Bond efficiency (Parra-Montesinos et al. 2005)

This test gave very good motivation on the possibility of eliminating joint transverse reinforcement while preventing joint damage through the use of ECC. However, no comparison can be made between ECC and RC beam-column sub-assemblages as no RC control specimen was prepared in this study.

2.4.4.2 Yuan's test

Yuan et al. (2013) tested six external beam-column joints under reversed cyclic loading. Details of the beam-column joints incorporating ECC can be seen in **Fig. 2.14**. ECC was incorporated in the joint core and adjacent beam and column plastic regions. All specimens were designed to strong-component-weak-joint approach, they had similar design of steel reinforcement for beam and column. For some specimens, joint stirrups were not provided.

Table 2.5 lists the details of each specimen. Two concrete specimens were prepared (S1 and S2) with and without joint stirrups. Among four ECC joints (S3-S6), the ratio of transverse reinforcement in the joints varied at 0%, 0.69% and 1.04%. The effect of column axial load was studied by deliberately increasing the axial load from 20 to 30% of column load-carrying capacity to one of the ECC joints (S5). **Fig. 2.15(a)** shows the test set-up, in which the column was placed in horizontal direction and the beam in vertical direction. The column was simply supported and a lateral cyclic load was applied through a hydraulic actuator mounted onto the strong wall. Column axial load was applied through a hydraulic jack. Loading history is shown as **Fig. 2.15(b)**, based on load-control method before yielding of steel reinforcement and displacement-control method after yielding. The lateral yielding load P_y was estimated based on the materials properties of the cross-section. The load was increased at $0.05P_y$ prior to reach yielding. Upon attaining yielding, yielding displacement Δ_y was measured. Thereafter, displacement-control at displacement level of $2\Delta_y$, $3\Delta_y$, $4\Delta_y$ and so on was applied until the specimens failed.

Fig. 2.14: Beam-column joints incorporating ECC (are units are in mm)
(Yuan et al. 2013)

Table 2.5: Details of the tested specimens (Yuan et al. 2013)

Specimen	Material in the joint	Transverse reinforcement ratio in the joint (%)	Axial Load (kN)
S1	Concrete	0	350
S2	Concrete	0.69	350
S3	ECC	0	350
S4	ECC	0.69	350
S5	ECC	0.69	525
S6	ECC	1.04	350

Fig. 2.16 shows hysteresis response of the tested specimens. Results revealed that ECC specimen without joint stirrups (S3) performed better in terms of load capacity, ductility and energy dissipation compared with the concrete specimen with joint stirrups (S2) as shown in **Figs. 2.16 (a and b)**. However, brittle joint shear failure was unavoidable in this ECC specimen. Similar failure was observed in concrete specimens S1 and S2, as indicated in **Fig. 2.17**. It is noteworthy that obvious pinching of hysteresis curve can be seen from **Figs. 2.16 (a and b)** due to joint shear failure. For ECC specimen with joint stirrups (S4), a more ductile failure was observed in addition to greater load capacity, ductility and energy dissipation. Final failure for S4 was caused by tensile rupture of longitudinal reinforcement, with a full plastic hinge developed in the beam. Moreover, results also demonstrated that increasing the amount of joint stirrups and column axial load in ECC specimens did not significantly improve hysteresis response of the ECC specimens (**Fig 2.16(d)**).

(a) Test set-up

(b) Loading history (unit of load in kN; unit of displacement in mm)

Fig. 2.15: Test set-up and applied loading scheme (Yuan et al. 2013)

Fig. 2.16: Hysteresis response (Yuan et al. 2013)

Fig. 2.17: Failure modes (Yuan et al. 2013)

2.4.4.3 Qudah's test

The effect of reducing the number of stirrups in the joints and adjacent beam and column plastic zones for beam-column sub-assemblages incorporating ECC was studied by Qudah and Maalej (2014). Nine one-third scale specimens designed to high seismicity zone were tested under reversed cyclic loading without column axial load. **Fig. 2.18** shows the details of a typical specimen. Longitudinal bars for columns were 12T10. For beams, longitudinal bars of 3T12 and 2T10 were provided at the top and bottom layers. For transverse reinforcement in the joint, beams and columns, 6 mm diameter of links were used. Results indicated that the load-carrying capacity and energy dissipation of ECC enhanced specimens were significantly improved compared with the control specimen made of concrete. Joint shear failure was observed in the control specimen as the demand of shear stress was slightly greater than the limit specified by the joint ACI-ASCE (i.e. $1.25\sqrt{f'_c}$). However, the failure mode has changed to beam hinging for all ECC specimens.

Fig. 2.18: Details of beam-column joint (all units are in m)
(Qudah and Maalej 2014)

This test programme concluded that the most effective way for incorporating ECC and eliminating transverse reinforcement is as follows:-

- (i) 100% stirrups were eliminated in the joint core, and 50% stirrups were eliminated in adjacent beam and column plastic zones with ECC incorporated in the joint core and adjacent beam and column plastic zones (**Fig. 2.18(b)**).
- (ii) 100% stirrups were eliminated in the joint core, while 50% stirrups were removed in adjacent beam plastic zones with ECC incorporated in the joint core and adjacent beam plastic zones.

Despite the good test results obtained by Qudah and Maalej (2014), there was a lack of information in this study, for instance, ECC tensile properties that contributed to the enhanced performance of the joints was not stated. Besides, anchorage bond of reinforcement in ECC beam-column joints was not addressed.

2.4.4.4 Zhang's test

In Zhang's test, three beam-column sub-assemblages representing a rigid frame of a railway bridge were tested under cyclic loading. ECC was used not only in the joint core, but also throughout the beam. All specimens were designed according to strong-column-weak-beam concept with the ratio of column flexural capacity to beam flexural capacity equal to 1.67, calculated base on reinforced concrete structures. Details of the specimens are shown in **Figs. 2.19 and 2.20**. There were no shear stirrups provided in the joint core of the three specimens. TJ-1 had shear stirrups in the beam and the column while TJ-2 had no stirrups in the beam with ECC cast in the whole span of beam as shown in **Fig. 2.20 (a and b)**. All the shear stirrups in the beam and the column were removed when ECC was used to cast the whole unit of TJ-3.

Fig. 2.21 shows the test set-up, in which the column was pinned to a strong floor and two roller supports were fixed near the beam end to simulate inflection point in a frame under a seismic event. A displacement-control method was used to apply cyclic loading; a total of 36 reversed cycles ranging from drift ratio of 0.5% to 6.0% were included. No axial load was applied in this test regime.

Fig. 2.19: Typical specimen of external beam-column joint (Zhang et al. 2015)

Fig. 2.20: Layout of specimens (all units are in mm) (Zhang et al. 2015)

Fig. 2.21: Test set-up (Zhang et al. 2015)

Fig. 2.22 shows the failure modes of the tested specimens. It was observed that TJ-1 and TJ-2 had failed in a similar manner, i.e. beam flexural with the forming of plastic hinges in the beams. No crushing of concrete was detected. But for TJ-3, severe crushing of the top and the bottom portion of beam cover occurred at the maximum load (**Fig. 2.22(c)**) due to inadequate distribution of Polypropylene (PP) fibers in the ECC mix. **Fig. 2.23** shows the pull-out of beam longitudinal bars at the far face of the column, due to the slip between beam longitudinal bars and ECC matrix in the joint. TJ-3 demonstrated better bond among the threes specimens; due to limited space between the beam bars and the column interface has weakened the bonding between reinforcement and ECC matrix in other specimens (Zhang et al. 2015). Hysteresis response revealed that TJ-2 performed better than TJ-1 in terms of ductility and energy dissipation despite no shear reinforcement in the beam. Obvious pinching was observed in TJ-1 due to rupture of beam top bars (**Fig. 2.24(a)**). It is noteworthy that TJ-1 achieved the greatest load capacity in positive loading direction due to its enhanced compressive strength of both concrete and ECC as indicated in **Table 2.3**. Overall, this test has shed light on the possibility of ECC to replace transverse reinforcement in the whole beam-column sub-assemblages, and yet the replacement was not only limited to the joint core. However, the behaviour of bond between reinforcement and ECC is still unclear. Therefore, more investigations are needed to close this gap.

(a) TJ-1

(b) TJ-2

(c) TJ-3

Fig. 2.22: Failure modes (Zhang et al. 2015)

(a) TJ-1

(b) TJ-2

(c) TJ-3

Fig. 2.23: Pull-out of beam bars (Zhang et al. 2015)

(a) TJ-1

(b) TJ-2

(c) TJ-3

Fig. 2.24: Hysteresis response (Zhang et al. 2015)

2.4.4.5 Said's test

Said and Razak (2016) tested two units of exterior beam-column joints. ECC was incorporated in one of the specimens, in the same configuration with Yuan's test.

Fig. 2.25 shows the details of the specimen design in which no stirrups were provided for both specimens. A constant axial load of 20% was applied in this test. As expected, ECC specimen behaved in a more ductile manner than the concrete specimen as shown in **Fig. 2.26**. ECC specimen has delayed the failure up to a drift ratio of 6.0%, instead of failing at a drift ratio of 4.5% in the concrete specimen. Shear failure occurred in the concrete joint but not in the ECC joint. However, some essential results were not reported such as joint shear stress. The compressive strength of both concrete and ECC were also not stated. The enhancement in higher load capacity and energy dissipation could be due to the effect of greater compressive strength of ECC compared with concrete.

Fig. 2.25: Details of specimen (Said and Razak 2016)

Fig. 2.26: Hysteresis response (Said and Razak 2016)

So far this section focuses on the experimental test programmes. The behaviour of these beam-column joint specimens can be modelled using component-based joint method as described in the next section.

2.5 Component-based Joint Model

Component-based joint model can facilitate the modelling of moment-resisting frame. This method has been incorporated in Eurocode 3, Part 1-8 - design of joints (BSI 2005). According to Jaspart (2000), the originality of the component method is to consider any joint as a set of individual basic components and the approach requires the following steps: (1) identification of active components for the joint; (2) evaluation of respective mechanical characteristics of each component; and (3) assembly of all the components.

Having confirmed that most of the brittle joint failures in beam-column sub-assemblages subjected to cyclic actions arose from shear and anchorage bond-slip, Youssef and Ghobarah (2001) developed a beam-column joint model which accounted both of these behaviour. **Fig. 2.27** shows the joint model, beams and columns are treated as elastic elements. The joint core is modelled by four pinned rigid members, connected with two diagonal shear springs. The connection between the joint and the beam end or the column end, so-called interface, allows bond-slip and crushing of concrete can be simulated through three concrete springs and three steel springs as shown in **Fig. 2.27**. Each of the concrete and the steel springs

represents the stiffness of an effective group of reinforcing bars in tension or compression and the effective concrete in compression.

Fig. 2.27: Joint model by Youssef and Ghobarah (2001)

Other than this, the joint model developed by Lowes and Altoontash (2003) explicitly represents three inelastic mechanisms that govern the joint behaviour, viz. (1) anchorage failure of the beam and the column longitudinal reinforcement embedded in the joint core; (2) shear failure of the joint core; and (3) shear-transfer failure at the joint-column and joint-beam interfaces (interface-shear failure). It comprises 13 one-dimensional components as shown in **Fig. 2.28** and each of them has an independent load-deformation response history. Eight bar-slip components simulate stiffness and strength losses associated with bond-strength deterioration for beam and column longitudinal reinforcement embedded in the joint core or shear panel. One shear-panel component simulates the strength and stiffness losses associated with shear failure of the joint core. Four interface-shear components are used to simulate loss of shear-transfer capacity at the interfaces of the joint core or shear panel. This super-element has four external nodes with a total of 12 external degrees of freedom (two translations and one rotation at each node) and four internal degrees of freedom. Since this joint model can be used to simulate beam-column joints, attention is drawn to the bond-slip response of steel rebars embedded in ECC joint core. More detailed constitutive models for each components will be discussed in Chapter 6.

Fig. 2.28: Joint model by Lowes and Altoontash (2003)

2.6 Summary

In the seismic design of a moment-resisting frame, the most important design method is to apply strong-column-weak-beam approach. Attentions should be paid to the joint detailing; limited joint shear stress and limiting size of rebars passing through joints to control bond-slip of the reinforcement embedded in concrete joints. Experimental tests showed that brittle joint shear failure would occur if one of the above criteria was not satisfied. Even if joint shear failure did not occur, serious pinching of hysteresis response was observed. The effect of pinching caused rapid deterioration of strength, stiffness and ductility of beam-column joints.

Although the advantages of using ECC in beam-column joints have been reported through previous research studies (Parra-Montesinos and Wight 2000; Parra-Montesinos et al. 2005; Qudah and Maalej 2014; Said and Razak 2016; Yuan et al. 2013; Zhang et al. 2015), there was still some missing information, which makes mechanical modelling of ECC joints rather difficult. For instance, in some of the studies (Parra-Montesinos et al. 2005), performance of beam-column joints with ECC joints could not be compared with normal concrete specimens as there was no control specimen in the test programme. The behaviour of bond between reinforcement and ECC was not addressed in some of the tests (Qudah and Maalej 2014; Said and Razak 2016), yet it is particularly important to evaluate the effect of ECC in enhancing bond-slip behaviour of beam longitudinal bars passing through the joints. In addition, some studies did not address the joint shear strength (Said

and Razak 2016; Yuan et al. 2013) of beam-column joints with ECC. This made assessment of shear strength of ECC joints challenging, and comparison of test results with code-defined limiting shear strength values could not be carried out. Furthermore, there are hardly any publications addressing the seismic design of beam-column sub-assemblages (made of ECC or concrete joints) based on Eurocode 8.

To address the aforementioned limitations in the existing literature, this study was carried out to gain a deeper knowledge on the behaviour of beam-column joints made of ECC when subject to cyclic loading. Previous research incorporated ECC material in the joint cores and throughout the plastic hinge length of beams and columns (Parra-Montesinos et al. 2005; Qudah and Maalej 2014; Said and Razak 2016; Yuan et al. 2013; Zhang et al. 2015). However, in this study, ECC material was only incorporated in the joint cores as ECC material is approximately 4 times more expensive than normal concrete and its use should be kept to the minimum.

Also, the beam-column joint model proposed by Lowes will be adopted in this study for simulating cyclic response of the beam-column joints. Therefore, the effect of ECC in material models for component such as bond-slip need to be incorporated to facilitate numerical simulations on the beam-column joints with ECC.

CHAPTER 3: EXPERIMENTAL PROGRAMME OF REINFORCED CONCRETE BEAM-COLUMN SUB- ASSEMBLAGES WITH ENGINEERED CEMENTITIOUS COMPOSITES

3.1 Introduction

Reviews on Chapter 2 revealed that incorporating ECC in the joint core and adjacent critical regions could improve structural performance of beam-column joints subject to seismic loading. Besides ECC material properties, design of specimens that meets the targeted structural performance is of great importance as it governs the structural performance of beam-column joints. In seismic design, there are three structural performance levels which are referred to as functionality requirements for an entire structure under a given seismic event:

- (i) Immediate occupancy (serviceability limit state)
- (ii) Damage control limit state
- (iii) Collapse prevention (survival limit state)

These performance levels correspond to different design limit states, serviceability depends on stiffness, damage control depends on strength and collapse prevention depends on ductility (Priestley and Paulay 1992). Since ECC is well-known of its superior ductility and multiple-cracking behaviours, therefore its performances in damage control and collapse prevention were emphasised in most of the previous tests. So far, ECC has been incorporated not only in the joint core but also in adjacent beam and column plastic hinge regions in beam-column joints as discussed in Section 2.4.4. However if the targeted structural performance is at lower level, ECC could be uneconomical to be incorporated in such condition due to high cost of PVA or PE fibres. Therefore, experimental tests on beam-column sub-assemblages incorporating minimal amount of ECC were conducted in this study viz. ECC was only used in the joint core region of the beam-column sub-assemblages.

3.2 Objectives of Experimental Programme

This study aimed to use the minimal amount of ECC in enhancing structural performance of interior and exterior beam-column sub-assemblages subject to seismic loading. The objectives of the experimental programme comprise the following:

- (1) To investigate the structural behaviour of the beam-column sub-assemblages incorporating ECC and subject to cyclic loading;
- (2) To evaluate the strength and ductility of the beam-column sub-assemblages incorporating ECC and subject to cyclic loading;
- (3) To analyse the components that contribute to total drift of the beam-column sub-assemblages incorporating ECC;
- (4) To evaluate the joint shear strength and damage tolerance of the beam-column sub-assemblages incorporating ECC;
- (5) To study the bond behaviour of beam longitudinal steel reinforcement passing through the ECC joints;
- (6) To study the performance of ECC in specimens with non-seismic detailing;
- (7) To study the behaviour of ECC joint cores without transverse reinforcement; and
- (8) To study the effect of column axial load on global behaviour of beam-column sub-assemblages.

3.3 Design of Specimens

3.3.1 Prototype structure

Specimens were extracted from a typical moment-resisting frame (office building). A six-storey office building with 6 x 4 bay was selected as the prototype structure. Interior and exterior beam-column sub-assemblages at the first storey were chosen for this study because they represented the most critical condition of beam-column joints to withstand gravity and seismic action. The columns were spaced at 5400 mm in both directions and the typical storey height was 3600 mm. **Fig 3.1** shows

the plan and elevation views of the prototype structure. The sizes of beams and columns were 300 mm by 500 mm and 500 mm by 500 mm, respectively. Specimens were designed under both seismic and non-seismic conditions. Specimens with non-seismic detailing were designed for gravity loads in accordance with Eurocode 2 (BSI 2004), while specimens with seismic detailing were designed according to ductility class DCM (medium ductility), as specified in Eurocode 8 (BSI 2004). It is noteworthy that Eurocode 8 prescribes two limit states to be checked, viz. ultimate limit state (no collapse requirement) and damage limit state as mentioned in Section 3.1. Concrete C30/37 was used in the design of specimens. High yield deformed bars with nominal yield strength of 500 MPa were used as longitudinal reinforcement and mild steel bars with a nominal yield strength of 250 MPa were used as transverse reinforcement. Although Eurocode 2 does not allow this grade of steel reinforcement to be used, due to the size of beams (150 mm wide by 250 mm deep and 20 mm concrete cover), the author has decided to adopt mild steel bars as stirrups so that there was sufficient spacing between longitudinal bars for compaction purpose. The set of design calculations is included in Appendix A.

3.3.2 Model specimens

Due to the physical constraints of Protective Engineering laboratory in Nanyang Technological University, the beam-column sub-assembly specimens were scaled down to one-half of the original structure. However, the reinforcement ratio was kept the same as the prototype building. Thus, the dimensions of beams and columns were 150 mm by 250 mm and 250 mm by 250 mm, respectively. **Table 3.1** and **Table 3.2** show the reinforcement details of prototype and scaled-down model of beams and columns, respectively.

(a) Plan view

(b) Elevation view

Fig. 3.1: The prototype structure (all units are in mm)

Table 3. 1: Reinforcement details of prototype and model beams

Specimens	Top reinforcement	Bottom reinforcement	Mid-span stirrups	Beam end stirrups (critical region)
Prototype beam (Non-seismic design)	3H20 (0.70%)	2H20 (0.47%)	R12@200 (0.38%)	R12@200 (0.38%)
Model beam (Non-seismic design)	3H10 (0.72%)	2H10 (0.48%)	R6@100 (0.38%)	R6@100 (0.38%)
Prototype beam (seismic design)	3H25 (1.09%)	3H25 (1.09%)	R12@200 (0.38%)	R12@100 (0.75%)
Model beam (seismic design)	3H13 (1.21%)	3H13 (1.21%)	R6@100 (0.38%)	R6@50 (0.75%)

The value in brackets is reinforcement ratio.

Table 3.2: Reinforcement details of prototype and model columns

Specimens	Longitudinal reinforcement	Stirrups	Stirrups (critical region)
Prototype column (Non-seismic design)	8H20 (1.12%)	R12@200 (0.23%)	R12@200 (0.23%)
Model column (Non-seismic design)	8H10 (1.14%)	R6@100 (0.23%)	R6@100 (0.23%)
Prototype column (seismic design)	8H25 (1.75%)	R12@200 (0.23%)	R16@100 (0.80%)
Model column (seismic design)	8H13 (1.93%)	R6@100 (0.23%)	R8@50 (0.80%)

The value in brackets is reinforcement ratio.

3.3.2.1 Interior beam-column sub-assemblages

In order to study the effect of ECC in mitigating brittle joint failure, the requirement of strong-column-weak-beam as mentioned in **Table 2.1** was not fulfilled in this series of specimens to facilitate joint failure. The ratio of column flexural strength to beam flexural strength ($\frac{\sum M_{Rc}}{\sum M_{Rb}}$) is estimated as 1.0 for all specimens. The actual value of flexural strength ratio will be presented in Chapter 4. **Table 3.3** shows the details of the four interior beam-column sub-assemblages. Both seismic and non-seismic designs of specimens were adopted in this series of tests. **Fig. 3.2** shows the

detailing of the interior beam-column sub-assemblages. In the notations, CIJ denotes concrete interior joint; EIJ represents ECC interior joint; S and NS refer to seismic and non-seismic design, respectively; and the numerals 1 and 0 are indications of joints with and without joint stirrups, respectively. In specimens CIJ-NS-0 and EIJ-NS-0 with non-seismic design (**Fig. 3.2(a)**), stirrups were not provided in the joint cores. In CIJ-S-1 with seismic design, stirrups were provided along with conventional concrete in the joint, with closer spacing of stirrups at the column ends, and greater steel contents in both beams and columns. In specimen EIJ-S-0, ECC was used in the joint with no stirrups so that the effectiveness of pure ECC in resisting shear force could be examined under cyclic loads (**Fig. 3.2(b)**).

For CIJ-NS-0 and CIJ-S-1 specimens, there was only one stage of casting. However, two stages of casting were adopted for specimens with ECC in the joint (EIJ-NS-0 and EIJ-S-0). Concrete beams and columns (not including joint core) were first cast and cured for seven days, then ECC was cast into the joint core during second stage. Before pouring ECC into the joint core, all cold joint interfaces were intentionally roughened to approximately 3 mm roughness complying with Eurocode 2 (BSI 2004). This is to ensure adequate interface shear strength between structural members and the joint.

Table 3.3: Details of interior beam-column sub-assemblages

Specimen	Material in joint	Type of design	Joint stirrups (ratio in %)*	Remark
CIJ-NS-0	Concrete	Non-seismic	0	Control
EIJ-NS-0	ECC	Non-seismic	0	-
CIJ-S-1	Concrete	Seismic	4R8@50 (0.8%)	Control
EIJ-S-0	ECC	Seismic	0	-

*The value in the parenthesis is the geometric reinforcement ratio calculated from $A_s/(bs)$, where $b=250$ mm and $s=50$ mm.

(a) CIJ-NS-0 and EIJ-NS-0

(b) CIJ-S-1 and EIJ-S-0

Fig. 3.2: Detailing of interior beam-column sub-assemblages (all units are in mm)

3.3.2.2 Exterior beam-column sub-assemblages

A total number of four exterior beam-column sub-assemblages were tested under cyclic loading as shown in **Table 3.4**. In this series of tests, all specimens were

designed with seismic detailing and satisfied the requirement of strong-column-weak-beam according to Eurocode 8 (BSI 2004). By keeping the same steel reinforcement content in beams and columns as in interior beam-column sub-assemblages (**Fig. 3.2**), flexural strength ratio ($\frac{\sum M_{Rc}}{\sum M_{Rb}}$) was twice of that in IJ series due to the only one side of beam. As the effect of column axial load was not investigated in IJ series, therefore it is included in this series of test. With the presence of axial load, column developed a greater flexural strength and increased the flexural strength ratio compared to specimens without axial load.

Fig. 3.3 shows the detailing of the exterior beam-column sub-assemblages. To study the feasibility of eliminating shear reinforcement in the joint core when ECC is used, no joint stirrups were provided for one of the ECC specimens (EEJ-A-0) as shown in **Fig. 3.3(b)**. The other two specimens were exactly the same design, in which ECC was used in the joint core while the joint transverse reinforcement was kept (EEJ-A-1 AND EEJ-NA-1) as shown in **Fig. 3.3(c)**. It is noteworthy that the effect of axial load was studied through these two specimens. In the notations, CEJ denotes concrete exterior joint; EEJ represents ECC exterior joint; A and NA are refer to specimens with and without axial load, respectively; and the numerals 1 and 0 are indications of joint cores with and without stirrups, respectively.

The control specimen was monolithically cast using normal concrete as shown in **Fig. 3.3(a)**. The other three were ECC joints which involved two stages of casting, i.e. concrete beams and columns were cast prior to the ECC joint core (**Figs. 3.3(b and c)**). Thus, three cold joints were formed at the beam-column joint interfaces. The first cast took seven days of curing and hardening; all cold joint interfaces were intentionally roughened to approximately 3 mm roughness complying with Eurocode 2 (BSI 2004) to ensure adequate interface shear.

Table 3.4: Details of exterior beam-column sub-assemblages

Specimen	Material in joint	Axial load	Joint stirrups (ratio in %)*
CEJ-A-1	Concrete	Yes	4R8@50 (0.80%)
EEJ-A-0	ECC	Yes	0
EEJ-A-1	ECC	Yes	4R8@50 (0.80%)
EEJ-NA-1	ECC	No	4R8@50 (0.80%)

*The value in the parenthesis is the geometric reinforcement ratio calculated from $A_s/(bs)$, where $b=250$ mm and $s=50$ mm.

(a) CEJ-A-1

(b) EEJ-A-0

(c) EEJ-A-1 & EEJ-NA-1

Fig. 3.3: Detailing of exterior beam-column sub-assemblages (all units are in mm)

3.4 Test Set-up

To simulate the actual boundary conditions, test specimens were pinned at the column base and roller-supported at both beam ends. Details of the experimental

set-up for interior beam-column sub-assemblages are shown in **Fig. 3.4**. The column was connected to the testing floor through a pin support and both ends of the beam were restrained against vertical displacements by two load cells, by means of which vertical reaction forces could be measured. This type of support condition allows free lateral displacement of both beam ends but their vertical movement was restrained. Thus, it can simulate a moment pattern in sub-assemblages similar to that in a joint of a building frame (Priestley and MacRae 1996). A reversible horizontal load was applied to the column top by using hydraulic actuator as shown in **Fig. 3.4**. The test set-up for exterior beam-column sub-assemblages was similar to that for interior beam-column sub-assemblages, but with only one side of the beam connected to a load cell as indicated in **Fig. 3.5**. To apply axial load to exterior beam-column sub-assemblages, a hydraulic jack was installed at the column top with four high-strength steel rods that connected a top steel plate and the bottom pin support as shown in **Fig. 3.5**.

Fig. 3.4: Test set-up for interior beam-column sub-assemblages (all units are in mm)

Fig. 3. 5: Test set-up for exterior beam-column sub-assemblages

3.5 Instrumentation

Figs. 3.6(a and b) show the layout of linear variable differential transducers (LVDTs) for interior and exterior beam-column sub-assemblages, respectively. LVDTs were mounted in the plastic hinge regions to measure flexural deformations of the beams and columns. LVDTs were installed at 150 mm spacing as shown in **Fig. 3.6**. Besides, two diagonal LVDTs were installed at a distance of 200 mm in the joint core to measure relative shear distortion, as shown in **Fig. 3.6(a and b)**. Strain gauges were also mounted on longitudinal reinforcement in the beams and columns as indicated in **Fig. 3.7(a and b)**, so that the stress state of longitudinal bars in the beams and columns could be captured. It should be noted that more than two strain gauges were installed on the longitudinal bars passing through the joint core, so as to capture the strain profile and bond anchorage of beam longitudinal reinforcement. To study the stress state in joint stirrups, strain gauges were also mounted on the stirrups as shown in **Fig. 3.7(b)**.

(a) Interior beam-column sub-assemblages

(b) Exterior beam-column sub-assemblages

Fig. 3.6: Layout of LVDTs (all units are in mm)

(a) Interior beam-column sub-assemblages

(b) Exterior beam-column sub-assemblages

Fig. 3.7: Layout of strain gauges (all units are in mm)

3.6 Loading History

During the test, cyclic load was applied at the column top through a hydraulic actuator with displacement-control method. **Fig. 3.8** shows the cyclic loading scheme. It ranged from the minimum drift ratio of 0.5% (equivalent to 9 mm horizontal displacement) to the maximum ratio of 6.0% (equivalent to 108 mm horizontal displacement) or an even higher drift ratio until a specimen finally failed. Despite the inter-storey drift ratio up to 2.0% was allowed by Eurocode 8 in designing RC tall building structures, more loading cycles have been applied beyond this for the purpose of evaluating seismic performance of beam-column sub-assemblages up to failure. It should be noted that the drift ratio was defined as the inter-storey displacement divided by the column height. Two load cycles were repeated in each drift ratio to evaluate the loss of strength and stiffness of the joints. Therefore, a total of 24 cycles were achieved if a specimen was loaded up to a drift ratio of 6.0% as shown in **Fig. 3.8**.

Fig. 3.8: Loading history

3.7 Column Axial Load

The effect of column axial load ratio on the strength of joints has been studied by many researchers. A moderate or high axial load is expected to improve the force transfer within the joint (Priestley and MacRae 1996) as well as shear strength of the joints. However, in the study of Lin and Restrepo (2002), a column axial compression above $0.3f_c'A_g$ (where f_c' is the compressive strength of concrete and A_g is the gross area of column) was detrimental to the joint shear strength because diagonal compression field tended to lean towards the diagonals of the joint. This was also supported by Li et al. (2015), when the axial load of $0.2f_c'A_g$ was increased to $0.6f_c'A_g$, specimens lost approximately 17.6% of peak strength and approximately 50% of energy dissipation. In some cases, no axial load was applied to columns during experimental tests to simulate the worst scenario of a seismic event (Hakuto et al. 2000; Qudah and Maalej 2014; Restrepo et al. 1995; Zhang et al. 2015). A moderate axial load ($0.2 - 0.3f_c'A_g$) was employed in the test to represent the scenario where the building structures were subjected to combined gravity and seismic actions (Lai 2012; Said and Razak 2016; Yuan et al. 2013).

In this experimental programme, no axial load was employed to any of the interior beam-column sub-assemblages to facilitate joint failure as strong-column-weak-beam requirement was not strictly imposed in this series of test. However, for

exterior beam-column sub-assemblages, a constant moderate axial load of $0.3f_c'A_g$ was applied to all the exterior beam-column sub-assemblages except one ECC specimen (EEJ-NA-1) for the purpose of study the effect of no axial load in this series of test.

3.8 Summary

In this study, beam-column sub-assemblages were designed and detailed in accordance with the category of ductility class DCM (medium ductility) in Eurocode 8 (BSI 2004). Since ECC is superior in terms of ductility, the chosen class of ductility cannot be too low. Further more, non-seismic design of interior beam-column sub-assemblages incorporating ECC was tested in this study for possible adoption of this type of joint in local practice. This experimental programme involved two series of tests, viz. interior beam-column sub-assemblages and exterior beam-column sub-assemblages. This two series of specimens were prepared and constructed at different times, so they had different material properties. Test results on materials will be reported in Chapter 4. Based on the detailed test set-up and proper arrangement of instrumentation, structural behaviour of the beam-column sub-assemblages and the joints can be observed and investigated.

CHAPTER 4: EXPERIMENTAL RESULTS OF REINFORCED CONCRETE BEAM-COLUMN SUB-ASSEMBLAGES WITH ENGINEERED CEMENTITIOUS COMPOSITES

4.1 Introduction

Following the specimen design, test set-up and instrumentation introduced in Chapter 3, test results of interior and exterior RC beam-column sub-assemblages subjected to lateral cyclic load will be presented in this chapter. In the first series of test (IJ series), four specimens were constructed, in which ECC was only incorporated in the joint core of two interior RC beam-column sub-assemblages, whereas another two were completely made of conventional RC specimens. This series of test involved both non-seismic and seismic design of specimens. Whereas for the second series of test (EJ series), four specimens were design to seismic detailing. Three of them were exterior beam-column sub-assemblages with ECC in the joint cores and another one served as a control specimen which made of conventional RC. Seismic performance of the RC beam-column sub-assemblages incorporating ECC was evaluated through comparisons on the hysteresis response, failure mode, analysis of inter-storey drift contribution, joint shear stress, distribution of beam and column longitudinal bars strain, strains in joint stirrups and energy dissipation between sub-assemblages made of ECC and RC joints.

4.2 Interior Beam-Column Sub-assemblages (IJ series)

4.2.1 Materials properties

In the design of specimens, normal concrete grade C30/37 was chosen as design compressive strength. High strength deformed bars H10 and H13 were used for longitudinal reinforcement in columns and beams, whereas mild steel bars R8 and R6 were used as shear links in columns and beams as shown in **Fig. 3.2**. **Table 4.1** summarises the material properties of steel reinforcement and concrete. For each size of bars, three samples were tested under tensile test and the average value of tensile properties was obtained as indicated in **Table 4.1**. The average compressive

strength of concrete was obtained through compressive test on three concrete cylinders (150 mm diameter x 300 mm height). **Fig. 4.1** shows the typical stress-strain relationship of longitudinal reinforcement under tension.

The desirable material properties of ECC play an important role in the joint behaviour under seismic loads. In this study, ECC was tailored by using locally available ingredients, in which Portland cement, ground granulated blast-furnace slag (GGBS), silica sand, water, superplasticizer (SP) and polyvinyl alcohol (PVA) fibres were mixed. **Table 4.2** shows the mix proportions of ECC. The water-binder ratio of ECC was 0.27 and 60% of binder was GGBS. The volume fraction of PVA fibres with 39 μm diameter and 12 mm length was 2%. Material properties of ECC were obtained by means of compression and uniaxial tension tests. The average compressive strength of three 50 mm diameter by 100 mm long cylinders was 50.3 MPa. When subject to uniaxial tension, ECC dog-bone specimens with a cross section of 10.5 mm thick by 35.5 mm wide and a gauge length of 125 mm, exhibited moderate strain-hardening behaviour as shown in **Fig. 4.2**. The first cracking strength of ECC was 3.2 MPa. Thereafter, several cracks developed along the gauge length, leading to several drops of applied load prior to failure. The average tensile strain capacity of three ECC coupons was around 1.2%.

Table 4.1: Material properties of reinforcing bars and concrete (IJ series)

Material	Bar type	Yield strength (MPa)	Elastic modulus (GPa)	Ultimate strength (MPa)	Fracture strain (%)
Longitudinal bar H13	High strength	540	216.0	650	10.5
Longitudinal bar H10	High strength	515	190.7	610	8.0
Stirrups R8	Mild steel	250	199.5	350	53.4
Stirrups R6	Mild steel	225	195.0	365	35.0
	Compressive strength (MPa)	Splitting tensile strength (MPa)		Modulus of elasticity (GPa)	
Concrete	24.7	2.8		23.3	

Table 4.2: Mix proportions of ECC

Ingredient	Cement	GGBS	Water	Silica sand	SP	PVA fibre
Unit eight (kg/m^3)	574	860	387	287	3-3.5	26

Fig. 4.1: Stress-strain curve of reinforcing bars (IJ series)

Fig. 4.2: Direct tensile test of ECC

4.2.2 Test results and discussions

The flexural strength ratio ($\frac{\sum M_{Rc}}{\sum M_{Rb}}$), which represents the column flexural strength to beam flexural strength ratio, was calculated from actual material properties of concrete and steel reinforcement (**Table 4.3**). The calculated flexural strength ratio for all specimens was 1.03, which was below the minimum flexural strength ratio of 1.30 as specified by Eurocode 8. The concept of strong-column-weak-beam was not strictly adhered to in this series of test, to allow for failure of joints and to measure the ultimate joint shear strength.

Table 4.3: Flexural strength ratio (IJ series)

Specimen	Flexural strength ratio $\frac{\sum M_{Rc}}{\sum M_{Rb}}$
CIJ-NS-0	1.03
EIJ-NS-0	1.03
CIJ-S-1	1.03
EIJ-S-0	1.03

4.2.2.1 Hysteresis response and failure mode

By applying a reversible horizontal load to the column top, the behaviours of interior beam-column sub-assemblages were observed and summarised in **Table 4.4**. The maximum load and failure load were taken in the positive loading direction (push). It is notable that all the tests were terminated after the full cycle of that particular drift ratio when the residual load resistance of the specimen dropped to 85% from its maximum load capacity (Li et al. 2015; Yuan et al. 2013).

Table 4.4: Summary of the test result (IJ series)

Specimen	Maximum Load (kN)	Failure Load (kN)	Drift ratio (%)		Final failure mode
			Maximum load	Failure load	
CIJ-NS-0	26.18	21.00	3.5	6.0	Diagonal shear cracks in the joint core and crushing of concrete at column region above joint.
EIJ-NS-0	23.82	18.42	3.0	4.5	Fracture of rebar, crushing and spalling of concrete at beam ends adjacent to the joint. No internal joint failure.
CIJ-S-1	40.98	33.10	3.5	6.5	Diagonal crushing of concrete at joint core.
EIJ-S-0	47.38	38.24	4.0	6.5	Crushing and spalling of concrete at beam ends adjacent to the joint. No internal joint failure.

Fig. 4.3 shows the hysteresis loops of the non-seismic design specimens. The theoretical lateral strength (H_i) of the beam-column sub-assemblages corresponding to the calculated beam flexural strength, which based on measured material properties are shown in the hysteresis loops. As for control specimen CIJ-NS-0 with non-seismic design, only a few flexural cracks on the beams were initiated at a drift ratio of 1.0%. With increasing horizontal displacement at the column top, more cracks were formed on the top and bottom of the beams. The maximum horizontal load of 26.18 kN was attained when the drift ratio was 3.5%. The theoretical lateral strength was nearly approached in positive loading direction, but load capacity decreased after drift ratio of 3.5%. By the same time, more cracks appeared on the joint core. Crushing of concrete was observed in the compression zones of the beams at a drift ratio of 5.0%. Subsequently, spalling of concrete occurred at the left column region above joint due to the extension of joint diagonal cracks during drift ratio of 6.0%. Final failure of the joint was caused by widening of diagonal shear cracks in the joint core (**Fig. 4.3(a)**). This type of failure is expected for the beam-column joints without transverse reinforcement in the joint core especially when the joint shear stress is large, significant diagonal tension crackings in both directions would occur (Hakuto et al. 2000).

When ECC was used in the joint core, beam-column joint EIJ-NS-0 developed different behaviour from CIJ-NS-0 as shown in **Fig. 4.3(b)**. It achieved the maximum load capacity of 23.82 kN at 3.0% drift ratio. Theoretical lateral strength was not achieved in both directions due to the forming of interface cracks between beams and the joint cores. Thereafter, a sudden drop of the load took place during the second cycle of 4.0% drift ratio. It resulted from rebar fracture of beam top reinforcement at the right joint interface as shown in **Fig. 4.4(a)**. The horizontal load decreased to 18.42 kN at 4.5% drift ratio. Although the maximum loads in the following cycles were less than 85% of the load capacity, testing was continued to a drift ratio of 6.0% for comparison purpose. Hysteretic response in negative loading phase exhibited good ductility up to a drift ratio of 6.0% as observed in **Fig. 4.3(b)**. During the second cycle of negative loading at a drift ratio of 6.0%, fracture of top rebar was observed at the left joint interface (**Fig. 4.4(b)**), leading to significant pinching of the hysteretic curve, as shown in **Fig. 4.3(b)**. Crushing of concrete was observed at the compression zone of beams at left and right top joint interfaces as shown in **Fig. 4.3(b)**. The fracture of beam top longitudinal bars could possibly be caused by serious widening of cracks at the joint interface due to different materials in the beams and the joint. However, no major damage was observed in the joint core, except for a few hairline cracks. By using ECC in the joint core, the energy dissipation of the sub-assembly was improved considerably compared to that of concrete specimen CIJ-NS-0. Further details on energy dissipation will be seen in section 4.3.2.6.

(a) CIJ-NS-0

(b) EIJ-NS-0

Fig. 4.3: Load-displacement hysteresis loops and final failure modes for non-seismic design specimens

(a) Right joint interface

(b) Left joint interface

Fig. 4.4: Fracturing of rebar in EIJ-NS-0

Specimen CIJ-S-1 with seismic design developed a load capacity of 40.98 kN at a drift ratio of 3.5%, as shown in **Fig. 4.5(a)**. In addition to flexural cracks in the beams and the column, diagonal shear cracks were formed in the joint core during drift ratio of 2.0% onwards. The rapid increasing of diagonal cracks in both directions controlled the attainment of load capacity, thus its theoretical lateral strength was unable to reach. Also, severe pinching could be found in the hysteresis curve of this specimen. During the second cycle of 5.0% drift ratio, crushing of concrete occurred in the joint core. With increasing horizontal displacement at the column top, spalling of concrete was observed in the joint core during drift ratio of 5.5% as shown in **Fig. 4.6(a)**. Eventually, the specimen failed by diagonal crushing of compressive strut in the joint core at 6.5% drift ratio.

As for the specimen EIJ-S-0, even though transverse reinforcement was eliminated in the joint, the behaviour of EIJ-S-0 was substantially improved by using ECC joint. Compared to CIJ-NS-0, its load capacity was approximately 15% greater (**Fig. 4.5(a)**). The peak load of 47.38 kN was attained at a drift ratio of 4%, which was very closed to theoretical lateral strength in positive loading direction. Brittle joint failure was prevented in EIJ-S-0. Instead, crushing and spalling of concrete occurred at the beam ends adjacent to the joint at drift ratio of 5.5% onwards (**Fig. 4.6(b)**). In the joint core, multiple cracks were observed with limited crack width at the final stage, due to better ductility of ECC in comparison with conventional concrete.

(a) CIJ-S-1

(b) EIJ-S-0

Fig. 4.5: Load-displacement hysteresis loops and final failure modes for seismic design specimens

(a) At the joint core of CIJ-S-1
(DR=5.5%)

(b) At the beam ends adjacent to the
joint of EIJ-S-0 (DR=5.5%)

Fig. 4.6: Crushing and spalling of concrete for seismic design specimens

Both specimens with ECC in the joint core (EIJ-NS-0 and EIJ-S-0) demonstrated beam failure as can be observed in **Figs. 4.3(b) and 4.5(b)**, despite the ratio of column flexural to beam flexural was below 1.30. Therefore, the strong-column-weak-beam requirement in the design of beam-column joint can be relaxed if ECC is used in the joint core. For concrete specimens (CIJ-NS-0 and CIJ-S-1), brittle joint failure was observed as beams were designed to have comparable flexural strength with columns. As ECC possess good damage tolerant and multiple-crackings behaviour, thus joint failure can be prevented in the beam-column sub-assemblages with ECC. The effect of ECC in changing the failure mode of beam-column sub-assemblages is strongly demonstrated when specimens were designed to have low flexural strength ratio.

4.2.2.2 Analysis of inter-storey drift contributions

Total horizontal displacement at the column top was mainly contributed by several major components, namely, flexural deformations of beams, flexural deformations of columns and shear distortions of the joint. Contributions of the aforementioned components could be calculated from the measured deformations by LVDTs (see **Fig. 3.6**) based on the method and procedure employed in previous research (Hakuto 1995). The beam and column shear deformations are neglected as they were considerably small compared with the three major components (Ashtiani et al. 2014). Displacement Δ_{bf} due to flexural deformations of beams is given by:

$$\Delta_{bf} = \delta_{bf} \frac{h}{l} \quad (4-1)$$

$$\delta_{bf} = \Sigma(\theta_{b,i})(l'_{bi}) \quad (4-2)$$

$$\theta_{b,i} = \frac{\delta_{iT} - \delta_{iB}}{h_i} \quad (4-3)$$

where δ_{bf} represents the beam flexural deformations; h is the storey height of the column; l is the horizontal distance between the beam supports in IJ series ($0.5l$ is used for EJ series); $\theta_{b,i}$ is the rotation at section i (including fixed end); l'_{bi} is the distance measured from section i to the centre of beam end support; δ_{iT} and δ_{iB} represent the readings of LVDTs mounted at the top and bottom faces of beam section i , respectively; h_i is the distance between the top and bottom LVDTs at section i . All details can be found in **Fig. 4.7**.

Fig. 4.7: Determination of beam flexural deformation

Displacement Δ_{cf} due to flexural deformations of columns can be calculated from:

$$\Delta_{cf} = \delta_{cf} \quad (4-4)$$

$$\delta_{cf} = \Sigma(\theta_{c,i})(l'_{ci}) \quad (4-5)$$

$$\theta_{c,i} = \frac{\delta_{iR} - \delta_{iL}}{h'_i} \quad (4-6)$$

where δ_{cf} represents the column flexural deformations; $\theta_{c,i}$ is the rotation at section i (including fixed end); l'_{ci} is the distance measured from section i to the centre of column end support; δ_{iR} and δ_{iL} represent the readings of LVDTs at the right and left faces of column section i , respectively; h'_i is the distance between the left and

right LVDTs at section i . Displacement Δ_j contributed by shear distortion of the joint can be determined as:

$$\Delta_j = \gamma_j \left(h - h_b - \frac{h}{l} h_c \right) \quad (4-7)$$

$$\gamma_j = \gamma_1 + \gamma_2 = \frac{\Delta_1 - \Delta_2}{\sqrt{2} * a} \quad (4-8)$$

in which γ_j represents the joint shear distortion; h_b and h_c are the depths of the beam and the column, respectively; Δ_1 and Δ_2 are the readings of diagonal LVDTs (S1 and S2) mounted on the joint and a is taken as 200 mm as shown in **Fig. 4.8**.

Fig. 4.8: Determination of shear distortion at joint panel

It is noteworthy that the sum of the total horizontal displacement calculated from Eq. (4-1, 4-3 and 4-7) was not perfectly equal to the total imposed drift as shown in **Fig. 4.9**. This could be due to those unmeasured parameters such as shear deformations in beams and columns as well as flexural deformations of beams outside the plastic hinge regions. However, the relative contributions of flexural deformations of beams, flexural deformations of columns and shear distortions of the joint were calculated with respect to their sum which always ended up with 100% (Ashtiani et al. 2014). **Fig. 4.10** shows the relative contributions of the three components to total drift in IJ series. In all the four specimens, shear distortion of the joint contributed little to total drift prior to diagonal cracking. Total drift was mainly caused by flexural deformation of beams. Only CIJ-S-1 experienced severe joint damage as a result of crushing of diagonal strut and spalling of concrete in the joint. Thus, it can be seen that 40% of total drift resulted from the joint distortion during drift ratio of

6.0%, whereas flexural deformations of beams and columns were in smaller fraction (**Fig. 4.10(c)**). However, for ECC specimens (EIJ-NS-0 and EIJ-S-0), flexural deformations of beams contributed more than 50% of the total drift throughout the loading cycles (**Figs. 4.10(b and d)**). It agreed well with the observed failure modes of beam-column sub-assemblages with ECC as shown in **Fig. 4.3(b)** and **Fig. 4.5(b)**. Likewise, the contribution of joint shear distortion was reduced significantly (85-90%) during a drift ratio of 6.0% by using ECC in the joint core. Analysis of inter-storey drift contributions has confirmed that ECC considerably changed the brittle joint failure to beam flexural failure of beam-column sub-assemblages as discussed in Section 4.2.2.1.

Fig. 4.9: Contributions to the overall imposed drift (IJ series)

Fig. 4.10: Relative contributions to the total calculated drift (IJ series)

4.2.2.3 Joint shear stress

Horizontal shear force at the mid-depth of the joint core can be calculated based on the following formula (Parra-Montesinos et al. 2005):

$$V_j = \sum \frac{M_b}{jd} - V_c \quad (4-9)$$

where M_b is the moment of the beams framing into the column in the loading direction; jd is the distance between the internal tension and compression force resultants in the beams (normally taken as $0.9d$) and V_c is the column shear force. Furthermore, joint shear stress can be estimated as:

$$v_j = \frac{V_j}{b_j h_c} \quad (4-10)$$

where b_j is the effective joint width and h_c is the column depth. Shear distortion is obtained from Eq. (4-8) by referring to **Fig. 4.8**. The calculated maximum joint shear stress was compared with New Zealand Standard (NZS) (Standard 2006), Eurocode 8 (EC8)(BSI 2004) and Joint ACI-ASCE 352(ACI-ASCE 2002). **Table 4.5** gives the limiting nominal joint shear stress provided by NZS and Joint ACI-ASCE 352.

Table 4.5: Nominal joint shear stress by codes (IJ series)

Type of joints	NZS	Joint ACI-ASCE 352
Interior joints	$0.20f_c'$ or 10 MPa whichever less	$1.25\sqrt{f_c'}$

On the other hand, EC8 specifies the maximum beam-column joint shear stress for interior joints as:

$$v_j \leq \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}} \quad (4-11)$$

$$\eta = 0.6(1 - \frac{f_c'}{250}) \quad (4-12)$$

where v_j is the joint shear stress; f_{cd} is the design concrete strength of concrete ($f_{cd} = \frac{f_c'}{1.5}$); v_d is the normalised column axial load ratio; η is the reduction factor due to diagonal tension cracking; f_c' is the characteristic compressive strength of concrete. It is worth mentioning that the effective width of the joint as defined in Eq. (4-10) was determined according to the respective codes (**Table 2.2**). According to EC8 and NZS, effective width of the joint is taken as 250 mm. On the other hand, effective width is taken as 200 mm for joint ACI-ASCE 352. Notable in EC8, h_c is taken as the distance between extreme layers of column reinforcement (i.e. 200 mm) while $h_c = 250$ mm based on the other two codes. Therefore the calculated shear stress was identical according to EC8 and ACI-ASCE 352 due to the same effective area of the joint core.

Table 4.6 shows the ratio of maximum calculated joint shear stress to limiting joint shear stress by the three different codes. Among them, only the maximum shear stress of joint CIJ-S-1 exceeded the allowable shear stress limit specified in both NZS and Joint ACI-ASCE, as crushing of diagonal compression strut was observed in the joint core. Therefore, NZS and Joint ACI-ASCE gave a good prediction on joints shear strength for the tested specimen. Shear strain of 1.1% implied that this specimen had undergone severe joint damage with wide cracks as well as crushing and spalling of concrete as depicted in **Fig. 4.5(a)**. Notably, allowable joint shear stress based on EC8 was over estimated if compared to what have been estimated by NZS or Joint ACI-ASCE. This implied that Eq. (4-11) in EC8 may not be conservative for calculating the failure load of diagonal compressive strut under zero column axial load. For the other specimens, calculated maximum shear stresses were below the limits specified by three codes as diagonal compression shear failure was not observed in any of the joints. Obviously, ECC specimens demonstrated less severe shear distortion if compared to their counterparts. This can be seen from the level of damage on joint and the cracks width appeared as shown in **Figs. 4.3(b) and 4.5(b)**. Thus, it is proven that ECC joints possess damage tolerant behaviour under cyclic loadings which required no repair on the joint during post-earthquake (Parra-Montesinos et al. 2005).

Table 4.6: Ratio of calculated maximum joint shear stress to limiting joint shear stress (IJ series)

Specimen	Calculated maximum joint shear stress according to NZS $v_{j\max}$ (MPa)	Calculated maximum joint shear stress according to EC8 and ACI-ASCE 352 $v_{j\max}$ (MPa)	Strain during $v_{j\max}$ (%)	Limiting joint shear stress $v_{j\text{limit}}$ (ratio $\frac{v_{j\max}}{v_{j\text{limit}}}$)		
				NZS	EC8	Joint ACI-ASCE 352
CIJ-NS-0	3.43	4.28	0.51	4.93(0.70)	8.88(0.48)	6.21(0.69)
EIJ-NS-0	3.24	4.05	0.07	10.00(0.32)	16.09(0.25)	8.87(0.46)
CIJ-S-1	5.21	6.51	1.10	4.93(1.06)	8.88(0.73)	6.21(1.05)
EIJ-S-0	6.29	7.86	0.38	10.00(0.63)	16.09(0.49)	8.87(0.89)

4.2.2.4 Strain distribution along longitudinal bars

To satisfy equilibrium of forces acting at the joint core of an interior beam-column sub-assembly as shown in **Fig. 2.2(a)**, when positive loading (push) is applied at the top of the column (direction of pushing by actuator is shown in **Fig. 3.5**), forces in the beam top bars supposed to be in tension at the right joint interface and compression at the left joint interface. Meanwhile, forces in beam bottom bars supposed to be in compression at the right joint interface and tension at the left joint interface. **Fig. 4.11** shows the measured strain profile along top and bottom longitudinal bars in the beam of CIJ-NS-0 under positive loading. Initially, beam top reinforcement at the left joint interface was not in compression (see **Fig. 4.11(a)**) due to relatively low ratio of tensile reinforcement at the bottom and limited neutral axis depth. With increasing drift ratio, tensile strains started to develop, indicating possible debonding of beam longitudinal reinforcement. Similar observation was

detected for beam bottom reinforcement at the right joint interface (see **Fig. 4.11(b)**).

However for EIJ-NS-0, compression stresses were generated at the left joint interface of the beam top reinforcement (**Fig. 4.12(a)**) and at the right joint interface of the beam bottom reinforcement (**Fig. 4.12(b)**) prior a drift ratio of 2.0%. Compared with CIJ-NS-0, anchorage bond strength was improved in EIJ-NS-0 based on greater gradient of strain profile along the anchorage length before a drift ratio of 4.0%. Yielding of beam longitudinal reinforcement in ECC specimen occurred earlier than that in concrete specimen, possibly due to the presence of cold joint at the vertical interface. Strain reading could not be obtained at the right joint interface during a drift ratio of 4.0% and above, as fracturing of rebar had already occurred by then.

(a) Beam top bar

(b) Beam bottom bar

Fig. 4.11: Distribution of strain along beam longitudinal bars passing through the joint core of CIJ-NS-0

(a) Beam top bar

(b) Beam bottom bar

Fig. 4.12: Distribution of strain along beam longitudinal bars passing through the joint core of EIJ-NS-0

Fig. 4.13 shows the distribution of strain along beam longitudinal bars for CIJ-S-1. Generally, strains in beam bars were kept below yield strain. Beam top bar was just about to yield at the left joint interface during the final loading stage as seen in **Fig. 4.13(a)**, leading to diagonal crushing of the joint since flexural moment strength was unable to be developed without yielding of beam bars.

(a) Beam top bar

(b) Beam bottom bar

Fig. 4.13: Distribution of strain along beam longitudinal bars passing through the joint core of CIJ-S-1

For EIJ-S-0, beam bars in tension zone have yielded after drift ratio of 3.0% (**Fig. 4.14**) indicating the development of beam flexural strength which contributed to greater lateral load capacity. According to Tsonos (2007), the ability of members to develop their flexural strength before failing in shear is of great interest in seismic design of beam-column joints. A greater gradient of the strain profile along the

anchorage length in this specimen implied that ECC has improved slightly the bond strength up to a drift ratio of 3.0% if compared with concrete specimen. Larger strain values at left and right joint interfaces were obtained, possibly due to widening of cracks at the vertical cold joint interfaces as drift ratio was increasing. Consequently, concrete crushing at the beam ends occurred and led to the failure of this specimen.

(a) Beam top bar

(b) Beam bottom bar

Fig. 4.14: Distribution of strain along beam longitudinal bars passing

through the joint core of EIJ-S-0

Fig. 4.15 shows the typical strains of column left longitudinal reinforcement passing through the joint for both concrete and ECC specimens. For concrete specimens (CIJ-NS-0 and CIJ-S-1), column longitudinal reinforcement did not yield at failure (**Fig. 4.15(a)**). However, due to the presence of cold joint in ECC specimens (EIJ-NS-0 and EIJ-S-0), strains of column reinforcement yielded at the joint interface at a drift ratio of 3.0%. Away from the interface, column reinforcement was in elastic stage. Despite yielding of column reinforcement had occurred, ECC specimens demonstrated better bonding than concrete specimens. This can be seen from the greater slope gradient exhibited by the strain profile along the beam depth as shown in **Fig. 4.15(b)**.

(a) CIJ-NS-0

(b) EIJ-NS-0

Fig. 4.15: Distribution of strain along column left longitudinal bars passing through joint (IJ series)

4.2.2.5 Strain in joint stirrups

Fig. 4.16 shows the strain history of the joint stirrups for CIJ-S-1. Positive strain readings indicated that these stirrups transferred tensile forces and have yielded at about drift ratio of 4.0%. Strain profiles at A, B, C and D were rather symmetry about the centre of the joint core, indicating a good test set-up was achieved. Strain reading after this cycle could not be recorded. In subsequent loading cycles, the efficiency of stirrups in transmitting shear forces was reduced and this led to compressive strut failure of the joint as observed in **Fig. 4.5 (a)**.

Fig. 4.16: Strain history of joint stirrups for CIJ-S-1

4.2.2.6 Energy Dissipation

Energy dissipation was computed from the summation of the area enclosed by each cycle of the hysteresis loops. For comparison purpose, total cumulative energy dissipation at each cycle for each specimen was plotted up to 24 cycles (drift ratio of 6.0%) as shown in **Fig. 4.17**. Obviously for non-seismic design, EIJ-NS-0 exhibited greater cumulative energy dissipation than CIJ-NS-0 (**Fig. 4.17(a)**). This enhancement was about 76% during cycle no. 24, despite EIJ-NS-0 experienced early stage fracture of rebar and it had lesser load capacity. Whereas for seismic design (CIJ-S-1 and EIJ-S-0), total energy dissipated in both specimens were identical (**Fig 4.17(b)**). Therefore EIJ-S-0 did not demonstrate enhancement in total energy dissipation compared to its counterpart (CIJ-S-1). However, comparable energy dissipation after eliminating transverse reinforcement from the joint in EIJ-S-0 highlighted the effectiveness of ECC in resisting shear in joint core. Similar observation can be found in Zhang's test (Zhang et al. 2015), in which comparable energy dissipation was attained even after reducing the amount of transverse reinforcement in beam and column components of the exterior beam-column sub-assemblages when ECC was used in the particular components.

Fig. 4.17: Cumulative energy dissipation (IJ series)

4.2.3 Conclusions (IJ series)

The following conclusions can be drawn from the test results of interior beam-column sub-assemblages:-

- (1) Application of ECC in the joint core of interior beam-column sub-assemblages (EIJ-NS-0 and EIJ-S-0) significantly changed brittle joint shear failure mode to ductile beam flexural failure mode due to enhanced shear resistance in the ECC joint core compared with conventional concrete (CIJ-NS-0 and CIJ-S-1).
- (2) The ratio of column flexural moment to beam flexural moment as specified by Eurocode 8 for satisfying strong-column-weak-beam design concept can be relaxed if ECC is used in the joint core of beam-column sub-assemblages. Both ECC specimens (EIJ-NS-0 and EIJ-S-0) demonstrated beam failure mode despite the requirement of strong-column-weak-beam was not satisfied.
- (3) ECC was capable to reduce shear distortion of the joint due to its multiple cracking behaviour and superior ductility. Contribution of shear distortion to total drift was significantly decreased by 85-90% with the use of ECC in the joints compared to concrete joints. Thus, ECC demonstrated good damage tolerance for the interior joints.
- (4) Even though ECC specimens (EIJ-NS-0 and EIJ-S-0) showed improved bond behaviour with longitudinal bars passing through the joint core at the initial stage, bond deterioration occurred with widening of cracks at the joint interfaces as increasing drift ratio.
- (5) For beam-column sub-assemblages with non-seismic design, ECC improved energy dissipation of EIJ-NS-0 by around 76% but lateral strength was barely reduced due to premature fracture of beam top reinforcement.
- (6) Nearly the same energy dissipation capacities were obtained for both beam-column sub-assemblages with seismic design. However, ECC slightly increased the load capacity of the beam-column sub-assemblage

by around 15%, despite the elimination of transverse reinforcement in the joint core.

4.3 Exterior Beam-Column Sub-assemblages (EJ series)

4.3.1 Materials properties

Similarly, normal concrete grade C30/37 was chosen as design compressive strength in this series of specimens. Since all the specimens were designed to seismic detailing, only high strength deformed bars H13 was used for longitudinal reinforcement in columns and beams, whereas mild steel bars R8 and R6 were used as shear links in columns and beams as shown in **Fig. 3.3**. **Table 4.7** summarises the material properties of steel reinforcement and concrete. It is noteworthy that the control specimen (CEJ-A-1) was cast in a second batch as the original specimen had sustained untended damage during the test. Thus, it had different material properties of reinforcing bars and concrete as compared to the others. **Fig. 4.18** shows a typical stress-strain relationship of a steel bar H13, H13(c) denotes tensile stress-strain curve for the control specimen CEJ-A-1.

Table 4.7: Material properties of reinforcing bars and concrete (EJ series)

Material (specimen)	Bar type	Yield strength (MPa)	Elastic modulus (GPa)	Ultimate strength (MPa)	Fracture strain (%)
Longitudinal bar H13 (EEJ-A-0, EEJ-A-1 & EEJ-NA-1)	High strength	525	201.92	610	8.7
Longitudinal bar H13 (CEJ-A-1)	High strength	525	201.92	616	11.6
Stirrups R8 (EEJ-A-0, EEJ-A-1 & EEJ-NA-1)	Mild steel	380	237.50	540	35.1
Stirrups R8 (CEJ-A-1)	Mild steel	267	213.84	377	53.8
Stirrups R6 (EEJ-A-0, EEJ-A-1 & EEJ-NA-1)	Mild steel	380	253.33	545	38.0
Stirrups R6 (CEJ-A-1)	Mild steel	385	233.33	530	54.4
	Compressive strength (MPa)	Splitting tensile strength (MPa)		Modulus of elasticity (GPa)	
Concrete (EEJ-A-0, EEJ-A-1)	31.20	3.02		26.3	

& EEJ-NA-1)			
Concrete (CEJ-A-1)	40.12	2.67	29.8

Fig. 4.18: Stress-strain relationship of steel bar (EJ series)

The same ECC mix as shown in **Table 4.2** was used in this series of test. However, material properties of ECC were obtained by means of compression and bending tests. The average compressive strength calculated from three samples of 150 mm diameter by 300 mm long cylinders was 53.1 MPa. For bending test, ECC plates of dimensions 75 mm wide by 300 mm long by 12 mm thick were prepared and tested. Four-point bending at a 240 mm clear span was employed. **Fig. 4.19** shows a typical load-deflection curve of ECC plates. After the first cracking, ECC plates showed a significant hardening behaviour with increasing load. The deflection corresponding to the maximum load of ECC plates could be up to 7.3 mm. Based on experimental results of ECC plates under four-point bending, tensile strength and strain capacity of ECC were calculated through inverse methods (Qian and Li 2007; Qian and Li 2008). Finally, the effective tensile stress was calculated as 3.36 MPa and the strain capacity of ECC in tension was around 1.96%.

Fig. 4.19: Load-deflection curve of ECC plates under four-point bending

4.3.2 Test results and discussions

The calculated flexural strength ratio for exterior beam-column sub-assemblages is shown in **Table 4.8**. The strong-column-weak-beam requirement was satisfied in all specimens. Therefore beam is expected to fail before joint or column in EJ series.

Table 4.8: Flexural strength ratio (EJ series)

Specimen	Flexural strength ratio
	$\frac{\sum M_{Rc}}{\sum M_{Rb}}$
CEJ-A-1	4.59
EEJ-A-0	4.07
EEJ-A-1	4.07
EEJ-NA-1	2.05

4.3.2.1 Hysteresis response and failure mode

Table 4.9 shows a summary of the test result for exterior beam-column sub-assemblages. The maximum load was reported in both loading directions due to unsymmetrical of hysteresis response, unlike IJ series. However, the failure load was considered in the positive loading direction (push) only. Final failure modes of exterior beam-column sub-assemblages are listed in **Table 4.9**.

Table 4.9: Summary of the test result (EJ series)

Specimen	Maximum Load +ve/-ve (kN)	Failure Load (kN)	Drift ratio (%)		Final failure mode
			Maximum load	Failure load	
CEJ-A-1	28.31/23.87	22.13	3.5	7.0	Crushing and spalling of concrete near the beam end adjacent to the joint. Crushing of concrete cover at far face of the column.
EEJ-A-0	22.83/22.60	15.63	3.0	7.0	Crushing and spalling of concrete near the beam end adjacent to the joint.
EEJ-A-1	22.58/23.54	18.32	3.0	6.5	Crushing and spalling of concrete near the beam end adjacent to the joint.
EEJ-NA-1	25.73/20.84	20.48	4.0	8.0	Crushing and spalling of concrete near the beam end adjacent to the joint.

Fig. 4.20 shows the hysteresis loops of the four exterior beam-column sub-assemblages. Generally all specimens exhibited a stable behaviour up to a drift ratio of 6.0%. For EEJ-A-0 and EEJ-A-1, their hysteresis responses in positive loading direction (push) and negative loading direction (pull) were quite identical. However, hysteresis responses for CEJ-A-1 and EEJ-NA-1 were not as symmetrical since the maximum load attained in pushing direction was greater than that in pulling direction. All specimens attained the theoretical lateral strength (H_i) as shown in **Fig. 4.20**, indicating the ability of beams to develop their flexural strength and maintained the strength up to a certain drift ratio as shown in each hysteresis response.

It can be observed that CEJ-A-1 could achieve the highest load capacity among all, possibly due to relatively higher flexural strength ratio (**Table 4.8**). However, among the ECC joints, elimination of joint stirrups (EEJ-A-0) and zero axial load (EEJ-NA-1) did not significantly change the global response of the exterior beam-column sub-assemblages. All specimens failed in the beam flexural mode and, none of them has failed in the joint cores or column regions as shown in **Fig. 4.21**. Observations on the all specimens followed the sequence of (1) beam flexural cracks initiated at a drift ratio of 1.0 to 1.5%; (2) flexural cracks in beams increased and propagated to form inclined shear cracks after achieving the maximum load capacity from a drift ratio of 3.5% onwards; (3) widening of cracks resulted in crushing of concrete within a drift ratio of 4.0 to 5.5% and (4) spalling of concrete after severe crushing near the beam end adjacent to the joint core during a drift ratio of 5.0 to 6.5%. **Fig. 4.21** shows the final failure modes of the exterior beam-column sub-assemblages. Bond splitting cracks were detected at the top and bottom of the beam longitudinal bars during the later stage of loading cycles for all specimens (**Fig. 4.21**). For specimens subjected to column axial load, cracks were not found in the joint cores except few tiny cracks were formed at ECC joint with joint stirrups (EEJ-A-1). It is believed that $0.3 f'_c A_g$ of axial load has delayed formation of cracks within the joint core as demonstrated in others' tests (Hakuto et al. 2000; Li et al. 2015).

For ECC joint without column axial load (EEJ-NA-1), multiple fine cracks appeared in the joint but no damage was observed at the end of the test. Only a few flexural cracks were found in the column. However, these were not observed in other three specimens (CIJ-A-1, EEJ-A-0 and EEJ-A-1) due to relatively high flexural ratios compared to EEJ-NA-1 (see **Table 4.8**).

Fig. 4.20: Load-displacement hysteresis loops (EJ series)

Although all specimens failed in a similar manner, a different behaviour was observed in the control specimen (CEJ-A-1) towards the end of the loading cycles. Crushing of concrete cover at the far column face of this specimen was detected from a drift ratio of 6.0% onwards (**Fig. 4.22**). This phenomenon can be explained through **Fig. 4.23**. Concrete compression force, C_{c2}' acts within the thickness of the concrete cover needs to be transferred through the depth of the joint, thus increasing the transverse tension in the cover concrete along the depth of the joint core (Paulay and Park 1984). When combined with circumferential tension due to bond forces, spalling of concrete could happen in the worst case. Spalling did not occurred in

this specimen but crushing was inevitable. However, this type of failure was not observed in any of the ECC joints, possibly due to the confinement effect of ECC which provide lateral resistance against cover crushing and spalling (Li and Fischer 2002).

Fig. 4.21: Final failure modes (EJ series)

Fig. 4.22: Crushing of concrete cover at far column face (CEJ-A-1)

Fig. 4.23: Internal forces in exterior joint core (Paulay and Park 1984)

In this series of test, all specimens were designed to satisfy strong-column-weak-beam requirement with flexural strength ratio of 2.05 and above. Thus all specimens failed in the beams rather than joint or column. The improvement of ECC in exterior joints can be seen when concrete cover at far column face in ECC joints was prevented from crushing.

4.3.2.2 Analysis of inter-storey drift contributions

Horizontal displacements caused by three components were calculated according to the method as described in IJ series. **Fig. 4.24** shows the contributions of the three components to the total imposed drift. Generally for all specimens, beam flexural was the main contributor in total drift throughout the loading cycles. It conforms the observation of beam failure for all specimens. Beam flexural covered nearly 50% of the total drift during a drift ratio of 6.0% for all ECC joints, but it was 86% for CEJ-A-1. This percentage experienced a sudden jump from a drift ratio of 5.0% to a drift ratio of 6.0% due to greater rotation recorded by LVDTs. It could be attributed to the plastic hinge occurred in this zone, thus gave rise in the beam rotation during this loading cycle.

As can be seen from **Fig. 4.24**, contribution of column flexural in total drift was obviously less than beam flexural for all specimens, supported with the evidence of

no or very little flexural cracks appeared on columns. This is due to the fact that columns had greater flexural rigidities as compared to beams. Contribution of joint shear distortion was extremely small for all ECC joints including the control specimen. However, it can be seen that contribution of joint shear in EEJ-A-0 (**Fig. 4.24(b)**) was slightly greater than EEJ-A-1 (**Fig. 4.24(c)**).

It was found that the total sum of the calculated horizontal displacements was not perfectly equal to 100% as compared to the imposed drift. This is possibly due to shear deformations of beams which were not measured during the test while shear cracks was observed in the beams during late loading stage (see **Fig. 4.21**). However, relative contribution of each component ended up to 100% was computed as shown in **Fig. 4.25**.

(a) CEJ-A-1

(b) EEJ-A-0

(c) EEJ-A-1

(d) EEJ-NA-1

Fig. 4.24: Contributions to the overall imposed drift (EJ series)

Fig. 4.25: Relative contributions to the total calculated drift (EJ series)

4.3.2.3 Joint shear stress

The horizontal joint shear force and shear stress were evaluated through the same method as in IJ series. **Table 4.10** shows the limiting joint shear stress for exterior joints as specified by NZS and Joint ACI-ASCE 352. For EC8, limiting joint shear stress for exterior joints is taken as 80% of the joint shear stress calculated as in the interior joints (Eq. (4-11)).

Table 4.10: Nominal joint shear stress by codes (EJ series)

Type of joints	NZS	Joint ACI-ASCE 352
Exterior joints	$0.20f_c'$ or 10 MPa whichever less	$1.0\sqrt{f_c'}$

Table 4.11 shows the ratio of maximum joints shear stress to limiting joint shear stress by different codes. For all exterior beam-column sub-assemblages, the calculated maximum joint shear stresses were far below the shear stress limits specified by the three codes. Therefore, diagonal compression strut failure was not observed in any of those specimens. However, a slight enhancement of ECC in joint shear can be seen from the relatively low value of $\frac{v_{jmax}}{v_{jlimit}}$ for specimen EEJ-A-0, EEJ-A-1 and EEJ-NA-1 if compared with CIJ-A-1 as shown in **Table 4.11**.

The effect of joint stirrups was not significant in the demand of joint shear stress for ECC joints. As can be seen in **Table 4.11**, joint stirrups were able to reduce the joint shear demand by average 6% (compared EEJ-A-1 with EEJ-A-0) for the three codes. Without axial load, the maximum joint shear stress was increased about 16% by comparing EEJ-A-1 and EEJ-NA-1 from NZS and Joint ACI-ASCE 352. This can confirmed by more cracks can be observed in the joint core of EEJ-NA-1 (**Fig. 4.21(d)**). It is believed that the presence of axial load has slightly improved the shear strength of the joint.

Among the three codes, Joint ACI-ASCE 352 gives more stringent values on the permitted joint shear stress, but does not consider the effect of axial load. On the other hand, the demand of joint shear stress estimated by NZS was the lowest among all (**Table 4.11**). Joint shear stress demand in exterior joints was generally lower than in interior joints due to only one side of beam moment acting into the joint core. Therefore exterior joints are less vulnerable to joint shear failure. Therefore, the effect of ECC was not significant in improving joint shear behaviour due to low demand on joint shear in EJ series.

Table 4.11: Ratio of calculated maximum joint shear stress to limiting joint shear stress (EJ series)

Specimen	Calculated maximum joint shear stress according to NZS v_{jmax} (MPa)	Calculated maximum joint shear stress according to EC8 and ACI-ASCE 352 v_{jmax} (MPa)	Limiting joint shear stress v_{jlimit} (ratio $\frac{v_{jmax}}{v_{jlimit}}$)		
			NZS	EC8	Joint ACI-ASCE 352
CEJ-A-1	3.50	4.38	8.02(0.44)	6.25(0.70)	6.33(0.69)
EEJ-A-0	3.60	4.48	10.0(0.36)	9.14(0.49)	7.29(0.61)
EEJ-A-1	3.39	4.24	10.0(0.33)	9.14(0.46)	7.29(0.58)
EEJ-NA-1	3.87	4.83	10.0(0.39)	13.39(0.36)	7.29(0.66)

4.3.2.4 Strain distribution along longitudinal bars

Fig. 4.26 shows the measured strain profile along the top longitudinal bars in the beam for EJ series. Generally, yielding of beam top bars occurred during a drift ratio of 2.5-3.0% in all specimens. This agrees well with the ability of beam-column joints to develop their flexural strength after beam bar yielding as observed in **Fig. 4.20**. Strains in beam longitudinal bars were expected to increase with each cycle of loading as the maximum displacement of the specimens was subjected to increase. As demonstrated in previous test (Ehsani and Wight 1985; Kalogeropoulos et al. 2016; Tsonos 2007), pull-out or slipping of rebars was considered to commence if the maximum strains during two consecutive cycles of loading remained the same or decreased, as long as buckling of reinforcing bar had not taken place. Therefore to examine this, strain gauge readings at the critical beam-column joint interface known as BT3 as shown in **Fig. 3.7(b)** were monitored. **Fig. 4.27** shows the strain history of BT3. Only strain readings up to a drift ratio of 5.0% were captured because buckling of beam bars might occur after this loading cycle. It is believed

that buckling of beam longitudinal bars occurs after crushing of concrete cover. In this test, crushing of concrete was observed during a drift ratio of 5.0% and above. Obviously from **Fig. 4.27**, strain reading by BT3 increased with the increment of loading cycles for all ECC joints but strain reading remained in CEJ-A-1 during a drift ratio of 3.0% and above. Therefore, slippage of beam longitudinal bars was initiated during this loading cycle for concrete specimen (CEJ-A-1), and led to the sudden increment of beam rotation as demonstrated in **Fig. 4.24(a)**. Slippage of beam bars did not occur in ECC specimens due to more compatible deformations between reinforcement and ECC.

Fig. 4.26: Distribution of strain along beam top longitudinal bars passing through joint (EJ series)

Fig. 4.27: Strain history of strain gauge BT3

Figs. 4.28 and 4.29 show the distribution of strains in column left and right longitudinal reinforcement passing through the joint for CEJ-A-1 and EEJ-NA-1, respectively. For all specimens, column longitudinal reinforcement remained elastic throughout the test. Yielding of column longitudinal bar was not occurred in any of the specimens. With the presence of axial load, all recorded strains in column longitudinal bars were in compression as shown in **Fig. 4.28**. Those ECC joints subjected to axial load (EEJ-A-0 and EEJ-A-1) exhibited the same strain profile with concrete joint (CEJ-A-1). However, in ECC joint without axial load (EEJ-NA-1), strains distributed in tension and compression as demonstrated in **Fig. 4.29**. Strain values were relatively higher than other specimens, thus some flexural cracks were found in the column as shown in **Fig. 4.21(d)**.

Fig. 4.28: Distribution of strain along column longitudinal bars passing through joint core of CEJ-A-1

Fig. 4.29: Distribution of strain along column longitudinal bars passing through joint core of EEJ-NA-1

4.3.5.2 Strain in joint stirrups (*EJ series*)

Fig. 4.30 shows the average strain history of the joint stirrups for CEJ-A-1, EEJ-A-1 and EEJ-NA-1. Average strain value was obtained from the two readings recorded in strain gauges mounted on a same layer of joint stirrups. Positive strain readings indicated that these stirrups transferred tensile forces in the joint cores. As can be seen from **Fig. 4.30(a)**, horizontal joint stirrups in CEJ-A-1 only sustained limited tensile strains up to a drift ratio of 7.0%. ECC joints (EEJ-A-1 and EEJ-NA-1) exhibited increasing tensile strains as drift ratio increased as shown in **Fig. 4.30(b and c)**. However, yielding strain was not achieved in any of the specimens at the end of the test. This implied that joint stirrups remained largely at the elastic stage up to failure of all exterior beam-column sub-assemblages, as horizontal joint shear forces were below the limiting joint shear strength as discussed in **Table 4.11**.

(a) CEJ-A-1

(b) EEJ-A-1

(c) EEJ-NA-1

Fig. 4.30: Strain history of joint stirrups (EJ series)

4.3.2.6 Energy dissipation

Fig. 4.31 shows the plot of total cumulative energy dissipation for each of the exterior beam-column sub-assemblages up to 24 cycles (drift ratio of 6.0%). Those specimens subjected to axial load (CEJ-A-1, EEJ-A-0 and EEJ-A-1) demonstrated greater total cumulative energy dissipation, it was around 30% of enhancement during a drift ratio of 6.0% if compared to specimen without axial load (EEJ-NA-1). As can be seen, total energy dissipated in EEJ-A-0 was closed to EEJ-A-1. This implied that joint stirrups can be totally eliminated from the joint core if ECC is used.

Fig. 4.31: Cumulative energy dissipation (EJ series)

The effect of ECC in enhancing total energy dissipation of beam-column sub-assemblages is not significant for this series of test. However, the fact that comparable energy can be obtained through the use of ECC in joint core without transverse reinforcement shed light of feasibility of ECC in avoiding congestion of the joint core.

4.3.3 Conclusions

The performance of exterior beam-column sub-assemblages with ECC was evaluated through hysteresis response, failure mode, analysis of inter-story drift,

joint shear stress, bar-slippage and energy dissipation. The effects of joint stirrups and column axial load were also justified. The following conclusions can be drawn from the experimental results.

- (1) The use of ECC in joint core did not improve load capacity of exterior beam-column sub-assembly as the failure mode was classified as beam flexural at the end of the test for all specimens. Hysteresis response was controlled by flexural capacity of the beam and columns, rather than joint strength under this mode of failure. Unexpectedly, the concrete specimen (CEJ-A-1) had higher compressive strength of concrete in beam and columns than in those ECC specimens, thus achieved higher load capacity. It is worthy to mention that this specimen was a re-built specimen as the original specimen was accidentally destroyed during the test (approximately a drift ratio of 2.0%).
- (2) In addition to beam flexural failure in all specimens, only concrete specimen (CEJ-A-1) experienced crushing of concrete cover at the far face of column. This phenomenon was prevented in ECC joints, because the confinement effect of ECC provide lateral resistance against cover crushing and spalling.
- (3) Contribution of joint shear distortion in total drift was insignificant for all specimens. Joint shear stress demand were well below limiting shear stress specified by codes as the ratio of flexural strength was relatively high. Therefore contribution of ECC in joints is rather limited for exterior beam-column sub-assemblies when the demand of joint shear stress was low.
- (4) Elimination of stirrups from ECC joint is feasible because ECC joint without stirrups (EEJ-A-0) performed closely to ECC joint with stirrups (EEJ-A-1) in terms of load carrying capacity, energy dissipation, joint shear stress and anchorage bond.
- (5) The effect of column axial load can be seen more obviously in the behaviour of joint shear. Joint shear demand was reduced by 16% for ECC joint with column axial load (EEJ-A-1) compare to ECC joint

without column axial load (EEJ-NA-1). Also, the presence of axial load has delayed the formation of cracks in joint cores.

- (6) Slippage of beam longitudinal bar at beam-column interface did not occur in any of the ECC specimens because of better bond between steel reinforcement and ECC. Bar-slippage in CEJ-A-1 imposed greater demand of plastic hinge rotation towards the end of loading cycles. However, bar-slippage was insignificant in affecting global load-displacement response of the exterior beam-column sub-assemblages.

4.4 Summary

In the first series of test, four interior beam-column sub-assemblages were designed to have joint shear failure by deliberately making the ratio of column flexural strength to beam flexural strength less than what has specified by codes according to strong-column-weak-beam approach. Also, no axial load was employed to facilitate joint shear failure. Results of the first series of test revealed that ECC significantly changed the failure mode of beam-column sub-assemblages from brittle joint failure to beam failure. The effect of ECC in non-seismic design specimens was greater than that in seismic design specimens in terms of total energy dissipation.

In the second series, the effect of ECC was rather limited in exterior beam-column sub-assemblages, which were designed to satisfy strong-column-weak-beam approach and had relatively low joint shear demand. Under the same failure mode of beam-column sub-assemblages made of concrete or either ECC in the joints, the effects of column axial load the joint stirrups were studied. Despite the performance of ECC was not significant in this series of test, but ECC is feasible to be used with elimination of joint stirrups in beam-column sub-assemblages subject to cyclic loading.

Experimental results for both series of test indicated a better bond anchorage can be achieved in beam-column sub-assemblages through the use of ECC in joint core. Bond stress of steel reinforcement embedded in ECC was hardly to be quantified at the post-yield stage from this experimental program. By far, bond-slip model for steel reinforcement embedded in ECC joint core is not well addressed. Therefore, a

series of component pull-out test is necessary to evaluate the bond stresses of steel reinforcement embedded in ECC at both elastic and post-yield stages. The behaviours of bond-slip for steel reinforcement embedded in ECC is particular important in the component-based joint modelling of beam-column sub-assemblages with ECC in the joint core.

CHAPTER 5: EXPERIMENTAL AND ANALYTICAL INVESTIGATION ON BOND-SLIP BEHAVIOUR OF DEFORMED BARS EMBEDDED IN ENGINEERED CEMENTITIOUS COMPOSITES

5.1 Introduction

Experimental investigation on bond-slip behaviour of reinforcement embedded in ECC is limited in literature. Currently available test data mainly focus on bond-slip models of longitudinal reinforcement in ECC flexural members such as beams (Asano and Kanakubo 2012; Bandelt and Billington 2014; Kunakubu and Hosoya 2015). In these cases, longitudinal splitting cracks could propagate along embedded reinforcing bars due to either inadequate concrete cover or close bar spacing. When the bond-slip model is used for longitudinal reinforcement in beam-column joints, significant underestimation of bond stress can be expected, as reinforcement in the joint is generally well-confined by thick concrete cover or closely spaced stirrups. Therefore, it is necessary to investigate the bond-slip behaviour of steel reinforcement embedded in beam-column joints. In order to fully understand the bond-slip behaviour of steel reinforcement embedded in ECC beam-column joints, two series of experimental tests were carried out on short and long reinforcement subject to pull-out actions. Bond stress in elastic stage was quantified through pull-out tests on short reinforcement while post-yield bond stress was obtained through long reinforcement. Analytical model for bond-slip is proposed based on experimental results to predict the force-slip relationship of long reinforcement either anchored in concrete or in ECC. Therefore this chapter presents experimental study and analytical model investigations of the bond-slip behaviour of steel reinforcement embedded in ECC.

5.2 Experimental Test

The objectives of pull-out tests were two-fold, viz. (a) to evaluate the effect of concrete or ECC on bond stress of short reinforcement and (b) to calculate the bond

stress along each steel segment of long reinforcement based on strain gauge measurements. From (a) and (b), one can gain physical insights into the interactions between either short or long reinforcement with ECC.

5.2.1 Specimen design

Fig. 5.1 shows the specimens design for both short and long reinforcement embedded in concrete or ECC blocks. **Table 5.1** summarises the details of steel reinforcement. In the first series, six steel bars with 13 mm diameter were pulled out from reinforced concrete and ECC blocks and their embedment length was only 65 mm (five times the rebar diameter), as shown in **Fig. 5.1(a)**. Thus, similar to the pull-out tests by Eligehausen et al. (1983), local bond-slip behaviour of short reinforcement was investigated at the elastic stage only. By increasing the embedment length, steel reinforcement may develop inelastic behaviour before failure occurs. After yielding of reinforcement, bond stress along the yielded steel segments reduces significantly due to inelastic elongation of reinforcement and shearing-off of concrete keys between steel lugs (Bigaj 1995; Shima et al. 1987). Therefore, post-yield bond stress of long reinforcement has to be studied as well. JSCE (2008) suggested that the minimum embedment length of reinforcement in ECC can be taken as twenty times the rebar diameter. Accordingly, to study the post-yield bond stress of reinforcement, steel bars of 13 mm nominal diameter were tested in the second series, and their embedment length was increased to 260 mm (twenty times the rebar diameter), as shown in **Fig. 5.1(a)**. Twelve specimens were designed in total, out of which six were embedded in conventional concrete and the other six in ECC. It is noteworthy that two 4.5 mm wide and 2.5 mm deep grooves were cut along the longitudinal ribs, as shown in **Fig. 5.1(a)**. Post-yield 2 mm gauge length strain gauges were mounted along the grooves on both sides of the reinforcement at intervals of 32.5 mm in the middle segment and 27.5 mm at the two end segments (see **Fig. 5.1(b)**).

Under pull-out loads, local cone failure of concrete would form at the active end of embedded reinforcement (Engström et al. 1998; Viwathanatepa et al. 1979), which would lead to bond deterioration and reduce the embedment length of reinforcement. The depth of the failure cone is generally around five times the diameter of

reinforcement (Soltani and Maekawa 2008). Therefore, to prevent the formation of failure cone of concrete, PVC pipes with 20 mm diameter and 65 mm length were provided at each end of short and long reinforcement and the effective embedment lengths of short and long reinforcement were kept as 65 mm (5ϕ) and 260 mm (20ϕ), respectively, as shown in **Fig. 5.1(a)**. Confining reinforcement consisting of 4H10 longitudinal bars and R6 transverse reinforcement at 100 mm centre-to-centre spacing was also placed in the concrete block to avoid potential failure.

Fig. 5.1: Details of embedded reinforcement

Table 5.1: Details of steel reinforcement

Specimen	Rebar diameter	Effective embedment length (grooved length)	Surrounding material
SC-1, SC-2, SC-3	13	65 (short)	Concrete
SE-1, SE-2, SE-3			ECC
LC-1, LC-2, LC-3		260 (Long)	Concrete
LE-1, LE-2, LE-3			ECC
LC-4, LC-5, LC-6			
LE-4, LE-5, LE-6			

5.2.2 Test set-up and instrumentation

(a) Schematic view

(b) Test set-up

Fig. 5.2: Set-up for pull-out tests

Fig. 5.2 shows the test set-up for reinforcement under pull-out force. Four PVC pipes were embedded in concrete blocks, through which four steel bolts were placed to connect the top and the bottom steel plates together. On the top steel plate a circular opening with a diameter of 80 mm (see **Fig. 5.2(a)**) was cut to reduce the effect of compression force on the bond stress of reinforcement. A similar opening was also made in the bottom plate to facilitate measurement of slip at the free end of

reinforcement. Projected steel reinforcement from the concrete block through the bottom plate was clamped by the testing machine to apply pull-out force to the embedded reinforcement. Applied load and axial displacements were directly measured by the testing machine. Besides, linear variable differential transducers (LVDTs) were installed at the loaded and free ends of the reinforcement to measure the slip of reinforcement relative to the concrete block, as shown in **Figs. 5.2(a and b)**. For long reinforcement with an effective embedment length of 260 mm, strain gauges were used to monitor the variations of steel strains along the embedment length (see **Fig. 5.1(b)**).

5.2.3 Material properties

Material tests were carried out on steel reinforcement H13 and ready mix concrete grade 30. **Table 5.2** summarises the material and geometric properties of original and grooved reinforcement.

Table 5.2: Material and geometric properties of steel reinforcement

Steel bars	Elastic modulus (GPa)	Yield strength (MPa)	Hardening modulus (MPa)	Ultimate strength (MPa)	Softening modulus (GPa)	Rupture stress (MPa)
H13	191.8	561	1232	658	14.5	456
	Effective circumference of grooved bars (mm)	Net area of grooved bars (mm ²)	Circumference of original bars (mm)		Area of original bars (mm ²)	
	31.0	105.9	42.0		127.8	
	Lug spacing (mm)		Lug inclination (°)		Lug area (mm ²)	
	9		62.0		13.9	

Fig. 5.3 shows a typical stress-strain curve of reinforcement in tension. Average compressive strength for three samples of 150 mm diameter by 300 mm long concrete cylinders was determined as 31.8 MPa. ECC was prepared by mixing water, ordinary Portland cement, ground granulated blast-furnace slag (GGBS), silica sand, superplasticizer, and micro-polyvinyl alcohol (PVA) fibres. The mix proportion of ECC was the same as shown in **Table 4.2**. The water-binder ratio of ECC was 0.27 and 60% of binder was replaced by GGBS. The volume fraction of PVA fibres with 39 µm in diameter and 12 mm in length was 2%. Material properties of ECC were also obtained by means of compression and uniaxial tension tests. The average compressive strength for three samples of 50 mm diameter by

100 mm long ECC cylinders was 53.2 MPa. The tensile stress-strain behaviour of ECC dog-bone specimens is indicated in **Fig. 4.2**.

Fig. 5.3: Stress-strain curves of steel reinforcement and ECC

5.2.4 Test result

5.2.4.1 Bond strength of short reinforcement

In the experimental tests, applied force and associated slip of short reinforcement were measured at the loaded end. By assuming constant bond stress distribution along the embedment length, bond stress at each load level was calculated by the applied load divided by the contact area of steel reinforcement and surrounding concrete or ECC. **Fig. 5.4** shows the bond stress-slip curves of short reinforcement under pull-out forces. At the initial stage, bond stress increased with slip of reinforcement at the loaded end. Nearly the same maximum bond stresses were obtained in the experimental tests of short bars embedded in conventional concrete, as shown in **Fig. 5.4(a)**. The average bond stress of three specimens was obtained as 17.1 MPa. However, the maximum force sustained by the steel reinforcement was much lower than the yield force, indicating that reinforcement remained at its elastic stage. Beyond the maximum bond stress, bond stress decreased with increasing slip at the loaded end and short steel reinforcement was pulled out from surrounding concrete as a result of insufficient embedment length. Similar bond-slip

curves were also obtained for reinforcement embedded in ECC. However, ECC substantially increased the maximum bond stress on reinforcement compared to concrete, as shown in **Fig. 5.4(b)**. The mean value for three specimens (with 5 ϕ anchorage length) was quantified as 24.9 MPa, nearly 45% greater than that of reinforcement in concrete. Higher bond stress between steel reinforcement and ECC resulted from the strain-hardening behaviour of ECC. After the formation of radial cracks around the reinforcement, ECC is able to carry the load continuously and to provide lateral confinement to the steel reinforcement. Thus, the peak bond stress in ECC was substantially increased compared to concrete specimens.

Fig. 5.4: Force-slip relationships of short reinforcement

To consider the effect of compressive strength of specimens, the maximum bond stress was normalised by $\sqrt{f_c'}$, where f_c' is the compressive strength of concrete or ECC cylinders. Thus, the values could be determined as $3.0\sqrt{f_c'}$ and $3.4\sqrt{f_c'}$ for reinforcement embedded in concrete and ECC, respectively. Compared to conventional concrete, ECC provided around 14% greater bond stress for short reinforcement. Besides, bond stress obtained from pull-out tests on short reinforcement embedded in concrete was significantly greater than that given by Bandelt and Billington (2014) and Eligehausen et al. (1983), possibly due to adequate concrete cover so that splitting cracks were prevented on the surface of the concrete block.

5.2.4.2 Bond-slip behaviour of long reinforcement

5.2.4.2.1 Force-slip curves

When the embedment length of reinforcement was increased to 260 mm (20ϕ), significantly different bond-slip behaviour was observed from that of short reinforcement. **Figs. 5.5(a and b)** show the force-slip relationships of long reinforcement. All the steel bars could develop their ultimate strength. Beyond the yield capacity of steel bars, long reinforcement experienced softening and eventually fractured at the onset of grooved sections (see **Fig. 5.6(a)**), instead of pull-out failure at elastic stage for short reinforcement. The surrounding ECC matrix remained intact during the whole pull out process, as shown in **Fig. 5.6(b)**. **Table 5.3** summarises the critical loads and corresponding slips at the loaded end of long reinforcement. Compared to conventional concrete, long reinforcement embedded in ECC developed nearly the same yield, ultimate and rupture forces, as shown in **Table 5.3**. However, associated slips of reinforcement in ECC were considerably smaller than those in concrete. For instance, the slip at fracture of reinforcement embedded in ECC was 9.29 mm, 18% smaller than that in concrete. The difference between the two sets of slips essentially resulted from higher bond strength between reinforcement and ECC.

Besides, slip at the free end of long reinforcement was also measured through LVDT (see **Fig. 5.2(a)**). **Figs. 5.5(c and d)** show the applied load versus free-end-slip curves of long reinforcement. It was observed that slip at the free end increased with applied load and attained its maximum value at the ultimate load capacity of reinforcement. The average values of maximum slips at the free end were around 0.75 mm in concrete and 0.69 mm in ECC. Thereafter, a reduction in applied load due to necking of steel bars gradually reduced the free end slip until fracture of reinforcement occurred near the loaded end. Test results indicate that the embedment length of steel reinforcement (20ϕ) is inadequate to ensure zero free end slip.

(a) loaded end slip of long reinforcement in concrete

(b) Loaded end slip of long reinforcement in ECC

(c) Free end slip of long reinforcement in concrete

(d) Free end slip of long reinforcement in ECC

Fig. 5.5: Force-slip relationships of long reinforcement

(a) Fracture of reinforcement

(b) Configuration of ECC block

Fig. 5.6: Failure modes of long reinforcement embedded in concrete and ECC

Table 5.3: Slips of reinforcement at critical loads

Specimen	Yield force F_y (kN)	Slip at F_y (mm)	Ultimate force F_u (kN)	Slip at F_u (mm)	Rupture force F_r (kN)	Slip at F_r (mm)
LC-1	65.6	1.44	75.8	5.77	57.2	10.14
LC-2	59.7	1.67	71.0	7.16	51.6	13.06
LC-3	61.7	1.21	73.4	5.27	52.7	11.78
LC-4	64.9	1.25	74.2	7.35	51.6	12.61
LC-5	61.8	1.15	73.8	4.53	50.2	10.02
LC-6	64.6	1.54	74.3	4.79	52.5	10.28
Mean value	63.1	1.38	73.8	5.81	52.6	11.32
Coefficient of variation	3.4%	13.6%	1.9%	18.8%	4.2%	10.9%
LE-1	62.5	0.95	74.4	3.64	49.6	9.55
LE-2	64.0	1.13	75.7	4.37	55.8	9.59
LE-3	58.4	0.91	71.9	3.88	54.1	8.39
LE-4	60.6	1.31	71.0	4.64	45.8	9.68
LE-5	60.3	1.11	71.1	3.89	46.5	8.99
LE-6	68.2	1.34	79.3	4.53	56.3	9.56
Mean value	62.3	1.13	73.9	4.16	51.4	9.29
Coefficient of variation	5.1%	14.4%	4.0%	9.0%	8.3%	4.9%

5.2.4.2.2 Steel strain profiles

In addition to measured forces and slips of reinforcement, steel strains at specific sections were recorded by strain gauges, as shown in **Fig. 5.1(b)**. **Figs. 5.7(a and b)** show the steel strain profiles along the embedment length of long reinforcement in ECC. At elastic stage, only limited embedment length of reinforcement was mobilised to transfer bond stress to surrounding concrete, as shown in **Fig. 5.7(a)**. When the yield strength was attained at the loaded end, nearly the full length of embedded reinforcement sustained tensile strain, with zero strain at the free end and the maximum strain at the loaded end. Furthermore, after the onset of inelastic behaviour at the loaded end, strains of yielded steel segments near the loaded end were several times greater than that of elastic segments, as shown in **Fig. 5.7(b)**. At the load capacity of long reinforcement, total length of inelastic steel segments was around 60 mm, and the remaining steel segments were at elastic stage. Thereafter the length of elastic steel segments remained nearly the same as that at the ultimate load of steel reinforcement. However, strains of inelastic steel segments were substantially increased due to localised elongation of reinforcement before fracture.

5.2.4.2.3 Bond stress profiles

With measured steel strains along the embedment length of long reinforcement, bond stress over each steel segment can be quantified by assuming a stress-strain model for steel reinforcement. In calculating the bond stress, a trilinear stress-strain model of reinforcement was employed (see **Fig. 5.3(a)**), as expressed in Eq. (5-1).

$$\sigma = E_s \cdot \varepsilon \quad \text{for } \varepsilon \leq \varepsilon_y$$

$$\sigma = \sigma_y + (\varepsilon - \varepsilon_y) \cdot E_h \quad \text{for } \varepsilon_y < \varepsilon \leq \varepsilon_u \quad (5-1)$$

$$\sigma = \sigma_u - (\varepsilon - \varepsilon_u) \cdot E_r \quad \text{for } \varepsilon_u < \varepsilon \leq \varepsilon_r$$

where σ and ε are the tensile stress and strain of reinforcement; E_s is the modulus of elasticity; σ_y and ε_y are the yield strength and yield strain of reinforcement; E_h is the hardening modulus of reinforcement; σ_u and ε_u are the ultimate stress and corresponding strain; ε_r is the strain at fracture; E_r is the softening modulus of reinforcement beyond its ultimate strength.

(a) Steel strains at elastic stage

(b) Steel strains at post-yield stage

(c) Bond stresses at elastic stage

(d) Bond stresses at post-yield stage

Fig. 5.7: Steel strain and bond stress profile along embedment length of long reinforcement in ECC

It was further assumed that the bond stress distribution along each steel segment is constant at the post-yield stage, so that the bond stress could be quantified within each segment. **Figs. 5.7(c and d)** show the bond stress profiles along the embedment length of reinforcement at various load levels. When the applied load was only 50% of the yield force of steel bars, bond stress at the free end of reinforcement was negligibly small and only limited embedment length was mobilised to transfer tensile stress between the steel reinforcement and surrounding ECC (see **Fig. 5.7(c)**). The maximum bond stress near the loaded end was around 25.6 MPa ($3.5\sqrt{f'_c}$) when the yield strength of reinforcement was attained.

It suggests that the maximum bond stress obtained from pull-out tests of short bars in ECC were also achieved for long reinforcement. Thereafter, bond stress at the loaded end was reduced to 8.4 MPa ($1.2\sqrt{f_c'}$) as a result of elongation of yielded steel segments, as shown in **Fig. 5.7(d)**. However, along the elastic steel segments, the maximum bond stress was still 12.9 MPa ($1.8\sqrt{f_c'}$). Beyond the ultimate strength of reinforcement, bond stress profile along the elastic steel segments remained almost unchanged, whereas a steep reduction of bond stress was observed along the yielded reinforcement, as shown in **Fig. 5.7(d)**, possibly due to localised necking of reinforcement at softening stage.

5.3 Analytical Bond-Slip Model

An analytical bond-slip model is proposed for reinforcement embedded in concrete and ECC in accordance with the experimental results. In the model, bond stress of reinforcement is related to slip at the loaded end if steel reinforcement exhibits elastic behaviour under pull-out forces. Once plasticity kicks in near the loaded end, post-yield bond stress calculated from long reinforcement can be utilised. Besides, by defining the trilinear stress-strain model for steel reinforcement (see **Fig.5.3(a)**), an attempt is made to study the bond-slip behaviour of reinforcement at softening stage due to necking of reinforcement near the loaded end.

5.3.1 Bond-slip model

At the elastic stage of steel reinforcement, bond-slip models proposed by Eligehausen et al. (1983) and Chao et al. (2010) are modified according to test results, as expressed in Eq. (5-2). The maximum bond stress is quantified through pull-out tests on short reinforcement embedded in concrete and ECC. As a result of better confinement provided by concrete and ECC, slip of reinforcement at the onset of maximum bond stress was increased to 1.5 mm. The frictional bond stress is assumed to be 37% of the maximum bond stress, as reported by Eligehausen et al. (1983) for short reinforcement embedded in concrete. **Table 5.4** includes the bond stress and slip of reinforcement anchored in concrete and ECC. The modified model agrees reasonably well with the bond-slip relationships of short reinforcement embedded in conventional concrete and ECC, as shown in **Figs. 5.4(a and b)**.

$$\begin{aligned}
\tau &= \tau_1 \left(\frac{s}{s_1} \right)^{0.5} \quad \text{for } s \leq s_1 \\
\tau &= \tau_1 \quad \text{for } s_1 < s \leq s_2 \\
\tau &= \tau_2 + (\tau_1 - \tau_2)(s_3 - s)/(s_3 - s_2) \quad \text{for } s_2 < s \leq s_3 \\
\tau &= \tau_2 \quad \text{for } s_3 < s
\end{aligned} \tag{5-2}$$

where τ_1 is the maximum bond stress of reinforcement; s_1 is the slip at the onset of maximum bond stress; s_2 is the slip at the end of maximum bond stress; τ_2 is the frictional bond stress; and s_3 is the slip at the onset of frictional bond.

Long reinforcement anchored in ECC developed inelastic behaviour close to the loaded end, as shown in **Fig. 5.7(b)**. Thus, it is necessary to evaluate the bond stress at the post-yield stage of reinforcement. In the experimental tests, average post-yield average post-yield bond stresses were determined as 4.2 MPa (around $0.75\sqrt{f'_c}$) and 5.7 MPa (around $0.78\sqrt{f'_c}$) for reinforcement in concrete and ECC, respectively. Besides, the post-yield bond stress can also be calculated from the frictional bond, if Poisson effect caused by contraction of cross section at the inelastic stage of reinforcement can be appropriately considered (Pochanart and Harmon 1989). It is reported that Poisson effect reduces the bond stress by 20 to 30% (Eligehausen et al. 1983). As a result, a reduction factor of 70% can be selected to quantify the post-yield bond stress from the frictional bond. Hence, corresponding values are determined as and for reinforcement anchored in concrete and ECC, respectively. These values are close to the experimental results of long reinforcement.

Table 5.4: Bond stresses and slips of reinforcement

Material	Maximum bond stress τ_1 (MPa)	Slip s_1 (mm)	Slip s_2 (mm)	Frictional bond stress τ_2 (MPa)	Slip s_3 (mm)	Post-yield bond stress τ_y (MPa)
Concrete	$3.0\sqrt{f'_c}$	1.5	3.0	$1.1\sqrt{f'_c}$	10.5	$0.8\sqrt{f'_c}$
ECC	$3.4\sqrt{f'_c}$			$1.3\sqrt{f'_c}$		$0.9\sqrt{f'_c}$

5.3.2 Calibration of analytical model

Fig. 5.8: Equilibrium and compatibility of steel segments

A nested iteration procedure is developed to estimate the force-slip relationship of long reinforcement. In the procedure, the embedded reinforcement is discretised into steel segments with a length of one rebar diameter. For each steel segment, relative slip at the two ends has to be assumed. Besides, it is further assumed that bond stress is constant along the steel segment, as shown in **Fig. 5.8(a)**. Correspondingly, bond stress can be calculated from the average slip at the centre of the segment in accordance with the proposed bond-slip model, and then total bond force acting on the segment can be computed, as expressed in Eq. (5-3). It is noteworthy that the steel force at the free end of reinforcement is zero. Hence, from the free end onwards, equilibrium of each steel segment can be established and steel strain at each section can be determined from the stress-strain model of reinforcement, as given by Eq. (5-1). Furthermore, compatibility of slips at both ends of the segment has to be satisfied (see **Fig. 5.8(b)**). Thus, slips at the ends of the steel segment can be calculated through Eq. (5-4).

$$\Delta F = \tau \pi \phi \Delta l \quad (5-3)$$

$$s_2 = s_1 + \frac{1}{2}(\varepsilon_1 + \varepsilon_2)\Delta l \quad (5-4)$$

where ΔF is the difference between forces at two ends of a steel segment; τ is the bond stress along a steel segment; ϕ is the diameter of a reinforcement; Δl is the length of a steel segment; ε_1 and ε_2 are the steel strains at both ends of a steel segment; s_1 and s_2 are associated slips.

Fig. 5.9: Comparison between experimental and analytical results of long reinforcement

In accordance with the proposed analytical model and bond stresses obtained from the experimental tests, the force-slip relationships of embedded steel reinforcement can be predicted. **Fig. 5.9** shows the analytical force-slip curves of long reinforcement in concrete and ECC. Comparisons between experimental and analytical results indicate that the analytical force-slip curves of reinforcement agree well with experimental results. Apparently, three stages can be categorised in the force-slip curves, namely, elastic ascending, hardening and post-peak softening stages. However, previous analytical studies on the bond-slip behaviour of reinforcement only focused on the elastic ascending and hardening stages, with little attention paid to the post-peak softening stage due to necking of steel reinforcement (Lowes et al. 2004; Yu and Tan 2010). Therefore, the ultimate slip of reinforcement was considerably underestimated, in particular, under large deformation state in which fracture of reinforcement may eventually occur.

5.3.3 Parametric study

Based on the proposed analytical bond-slip model in Section 5.3.2, a parametric study is conducted on the effect of embedment length on force-slip relationships of embedded reinforcement in concrete and ECC. In the study, four modes of failure, namely, elastic pull-out, post-yield pull-out, fracture with and without free end slip, are defined for steel reinforcement with various embedment lengths, as included in **Table 5.5**. Corresponding embedment lengths are calculated through the proposed model.

Table 5.5: Failure modes and embedment length of reinforcement

Material	Ratio of embedment length to rebar diameter			
	Elastic pull-out	Post-yield pull-out	Fracture with free end slip	Fracture without free end slip
Concrete	$L/\phi < 9$	$9 \leq L/\phi < 14$	$14 \leq L/\phi < 35$	$35 \leq L/\phi$
ECC	$L/\phi < 8$	$8 \leq L/\phi < 12$	$12 \leq L/\phi < 26$	$26 \leq L/\phi$

When the ratio of the embedment length to diameter of steel bars embedded in ECC is less than 8, reinforcement exhibits pull-out failure at elastic stage, similar to that observed in pull-out tests of short reinforcement. When the ratio is in the range of 8 to 12, reinforcement develops inelastic behaviour at the loaded end, but is eventually pulled out from surrounding ECC. By increasing the ratio beyond 12, fracture of reinforcement occurs at the loaded end. However, slip at the free end of reinforcement can still be mobilised due to insufficient embedment length, as discussed in pull-out tests of long reinforcement. A further increase in the ratio does not alter the failure mode of reinforcement, whereas slip at the free end of reinforcement is gradually reduced, leading to reduction in associated bond stress. The ratio has to be greater than 26 in order to fully eliminate the free end slip. Compared to ECC, the required embedment lengths of reinforcement in concrete have to be increased to 35 times its diameter, as shown in **Table 5.5**. Therefore, ECC can reduce the embedment length of steel reinforcement by 26% if free end slip is prevented.

5.3.4 Discussions

JSCE (2008) specifies the basic embedment length of reinforcement in ECC, in which bond stress of steel bars is assumed to be identical to that in concrete. If safety factor is not considered, the required embedment length of steel bar H13 can be determined as 26ϕ so as to develop its yield strength. Analytical results also show that steel reinforcement with such an embedment length can develop its ultimate tensile strength without slip at the free end. Thus, design values for embedment length of reinforcement are quite conservative compared to analytical results. Additionally, test results of long reinforcement in ECC indicate that steel bars with shorter embedment length could develop their ultimate strength, even though slip and bond stress at the free end of reinforcement were mobilised to resist the applied load. Analytical bond-slip model predicts the minimum embedment length of reinforcement as 12ϕ . However, slip at the free end is undesirable for embedded reinforcement, as it increases the total slip of reinforcement and reduces the energy dissipation capacity, in particular under cyclic loading conditions. Therefore, the embedment length of reinforcement in ECC needs to be at least 26ϕ so as to prevent slip at the free end. Moreover, the value should be further increased when the confinement condition of reinforcement is less than that in the experimental tests.

5.4 Conclusions

In this chapter, bond-slip behaviour of short and long steel reinforcement embedded in well-confined conventional concrete and ECC was investigated experimentally. Short reinforcement with embedment length of five times rebar diameter exhibited pull-out failure at elastic stage. However, long reinforcement fractured under pull-out loads, even though slip at the free end was mobilised as a result of inadequate embedment length. Comparison between concrete and ECC specimens indicates that ECC could increase the maximum local bond stress of short reinforcement by 14%. Besides, steel strains were measured by strain gauges mounted along the embedment length of long reinforcement. Associated bond stresses were calculated between adjacent strain gauges at each load stage.

An analytical bond-slip model was proposed in accordance with test results of short reinforcement featuring pull-out failure at elastic stage. Special attention was paid to the bond-stress of long reinforcement at post-yield stage. A nested iteration procedure was employed for embedded reinforcement under pull-out forces, in which elastic ascending, hardening and post-peak softening stages of reinforcement could be simulated. The model yields reasonably good predictions of force-slip relationships of long reinforcement embedded in concrete and ECC. Through the analytical model, the effect of embedment length on failure mode of reinforcement was studied and the required length for reinforcement to rupture without mobilising slip at the free end was quantified. Finally, design recommendations were made for embedment length of reinforcement in well-confined ECC.

CHAPTER 6: NUMERICAL SIMULATIONS ON SEISMIC RESPONSE OF RC BEAM-COLUMN SUB-ASSEMBLAGES WITH ENGINEERED CEMENTITIOUS COMPOSITES

6.1 Introduction

The component-based joint model as mentioned in Section 2.5 is employed to simulate seismic performance of the tested RC beam-column sub-assemblages either with or without ECC joints. ECC is shown to improve bond-slip behaviour of reinforcement embedded in beam-column joints from Section 4.2.2.4 and 4.3.2.4. This enhancement has to be incorporated in the bar-slip model to simulate beam-column sub-assemblages made with ECC material. Therefore, the improved bond-slip behaviour of ECC joints from pull-out tests as in Section 5.2.4.1 will be applied in the bar-slip model for beam-column joints made with ECC material. This chapter presents component-based joint modelling for RC beam-column sub-assemblages using OpenSees software. Constitutive models for all the components in the beam-column joint model are discussed in the following sections.

6.2 Numerical Simulation Using OpenSees

The component-based joint model developed by Lowes and Altoontash (2003) is well established in software OpenSees which is an object-oriented, open-source program to simulate the performance of structural systems subject to earthquakes (Mazzoni et al. 2006). **Fig. 6.1** shows a typical model of RC beam-column sub-assemblage subject to dynamic lateral cyclic loads and constant axial load at the top of the column. In the model, beams and columns (elements 1 through 8) are simulated using *Nonlinear beam-column elements* which assuming that plasticity spreads along the elements for interior beam-column sub-assemblages. However for exterior beam-column sub-assemblages, *beamWithHinges element* was used to simulate inelastic response of beams (element 6 or 8 in **Fig. 6.1**). In *beamWithHinges element*, a linear moment distribution is assumed and the integration scheme of two quadrature points within

the user-defined plastic length at the element ends and a single quadrature point at mid span (Mazzoni et al. 2006). This means the plasticity to be concentrated over the specified plastic hinge length. In this case, only one end of plastic hinge is assumed for element 8 (see **Fig. 6.1**). Therefore the non-plastic hinge length of this element is considered to be linear elastic. The plastic hinge length is further assumed to be half of the height of the member (Corley 1966). This assumption agrees reasonably well with observation during tests. Also, the joint core (element 9) is represented by a *beam-column joint element*. Both of the beam elements can be represented by a fibre section which contains reinforcement and concrete material properties.

The *beam-column joint element* (see **Fig. 2.28**) was discussed in Section 2.5; it can explicitly represent the anchorage failure of the bars passing through the joint core, shear failure of the joint core and shear-transfer failure at the joint interfaces. This element has in total 13 components (1 shear panel, 4 interface-shear springs and 8 bar-slip springs), in which their material models will be further discussed in Section 6.3.2.

Fig. 6.1: Typical model of an RC beam-column sub-assembly

6.3 Material Model

6.3.1 Nonlinear beam-column element

Concrete material response is simulated using *Concrete01 material* with zero tensile strength. Modified Kent and Park concrete model (Scott et al. 1982) is adopted to represent the compression behaviour of concrete. It consists of an ascending parabolic branch and a descending linear branch as shown in **Fig. 6.2**. The ascending parabola and the softening line are expressed in Eqs. (6-1) and (6-2), respectively.

Fig. 6.2: Modified Kent and Park concrete model

$$f_c = kf'_c \left[\frac{2\varepsilon_c}{k\varepsilon_0} - \left(\frac{\varepsilon_c}{k\varepsilon_0} \right)^2 \right] \quad \text{when } \varepsilon_c \leq k\varepsilon_0 \quad (6-1)$$

$$f_c = kf'_c [1 - Z_m(\varepsilon_c - k\varepsilon_0)^2] \quad \text{when } \varepsilon_c > k\varepsilon_0 \quad (6-2)$$

where f'_c is the maximum compressive strength of concrete; ε_0 is the strain of unconfined concrete corresponding to f'_c ($\varepsilon_0 = 0.002$); k is the confinement coefficient ($k = 1 + \rho \frac{f_y}{f'_c}$), k is taken as 1 for unconfined concrete; ρ is the ratio of the volume of transverse reinforcement to the volume of concrete core measured to the outside of stirrups. Thus, ρ is greater for transverse reinforcement arranged in closer spacing for the same size of rebar. Z_m is the decreasing ratio for confined

concrete after peak stress ($Z_m = \frac{0.5}{\varepsilon_{50u} + \varepsilon_{50h} - \varepsilon_0 k}$); f_y is the yield strength of transverse reinforcement in MPa;; ε_{50u} is taken as $\frac{3 + \varepsilon_0 f_c'}{f_c' - 1000}$; ε_{50h} is taken as $0.75\rho\sqrt{h/s}$ where h is the width of concrete core measured to the outside of stirrups and s is the centre to centre spacing of stirrups. It should be noted that both interior and exterior beam-column sub-assemblages used the same concrete model. **Fig. 6.3(a) shows the material model for steel reinforcement *Steel02 material* which is based on Giuffre-Menegotto-Pinto model (Menegotto 1973) with an isotropic strain hardening.** Strain hardening ratio (b), i.e. ratio between post-yield tangent (E_t) and initial elastic tangent (E_0), is taken as 0.0067 from actual material properties. R is a parameter which influences the shape of the transition from elastic to plastic state and it allows a good representation of the Bauschinger effect. R_0 is the value of the parameter R during the first loading whereas a_1 and a_2 are experimentally determined parameters to be defined together with R_0 (Filippou et al. 1983). In OpenSees's command manual (Mazzoni et al. 2006), recommended values of these parameters are $R_0 =$ between 10 and 20 (it is taken as 18.5 in the model); other isotropic hardening parameters ($a_1=0$, $a_2=0.4$, $a_3=0$, $a_4=0.5$) are assumed as in Lowes's study (Lowes and Altoontash 2003).

Fig. 6.3: Giuffre-Menegotto-Pinto reinforcing steel model (Filippou et al

1983)

6.3.2 Beam-column joint element

As described in Section 2.5, the beam-column joint element has three major components that have different material constitutive models. Lowes et al. (2003) have developed the constitutive relationships that enable users to define the load-deformation behaviour of the shear-panel component and bar-slip components as a function of material properties, joint geometry and reinforcement layout. However, due to the use of ECC in the joint core, those constitutive relationships have to be reviewed and calibrated to cater for the effect of ECC in simulations.

For the interface-shear component, it is assumed that flexural cracks at the beam-column joint interfaces would be closed upon load reversal and capacity of shear transfer across crack surface did not decrease. Shear transfer at the perimeter of the joint occurs through a compression-strut mechanism and it is assumed that this compression-strut mechanism has adequate strength and relatively high stiffness (Lowes et al. 2003). Thus modulus of elasticity, $E = 1 \times 10^7$ GPa is employed in the simulation. Therefore, it is further postulated that interface-shear spring will not degrade in strength and stiffness.

6.3.2.1 Constitutive Model for Shear Panel

Fig. 6.4 shows the constitutive model employed in the shear panel, identified as *Pinching4 material* in OpenSees. This model represents a pinched load-deformation response and exhibits degradation under cyclic loading. It is represented by multi-linear envelopes and tri-linear unload-reload paths as shown in **Fig. 6.4**. Calibration of this model requires 16 parameters to define the response envelope ($ePd_1, ePf_1, ePd_2, ePf_2, ePd_3, ePf_3, ePd_4, ePf_4, \dots$ as shown in **Fig. 6.4**); 6 parameters to define the unload-reload path ($rDispP, rDispN, rForceP, rForceN, uForceP, uForce$ as shown in **Fig. 6.4**); and 15 parameters (as in **Table 6.2**) to define three hysteretic damage rules (unloading stiffness degradation, reloading stiffness degradation and strength degradation).

Fig. 6.4: *Pinching4* material for shear panel (Mazzoni et al. 2006)

6.3.2.1.1 Response envelope

In Lowes's simulation (Lowes and Altoontash 2003), the Modified Compression Field Theory (MCFT) developed by Vecchio and Collins (1986) is employed to define the envelope of shear stress versus shear strain history of the joint core. For cyclic response, experimental data provided (Stevens et al. 1991) are used to define the unloading-reloading path and the hysteretic damage rules of the joint shear panel. Stevens' test was a continuation of Vecchio's model in which MCFT was employed for finite element implementation. However, MCFT is not suitable for joints without stirrups. MCFT is accurate only for joints with moderate-to-high joint transverse reinforcement ratio (Mitra and Lowes 2007). This is because in MCFT (Vecchio and Collins 1986), the assumption that the longitudinal and transverse reinforcing bars are uniformly distributed over the joint element is made, together with perfect bonding between concrete and reinforcing bars at the boundaries of the joint element. Therefore, a simple and unified joint shear model developed by Kim and LaFave (Kim and LaFave 2009) is employed instead. This model can be applied whether the joint core has stirrups or not. Their model can capture a wide range of different joint geometries, confinement by varying degrees of joint

reinforcement and beam longitudinal reinforcement, and concrete compressive strength. The envelope curves of the joint shear stress versus shear strain curves are developed by connecting key points such as cracking, yielding, and peak loads (Kim and LaFave 2009; LaFave and Kim 2011).

Generally in joint shear stress-strain response, the positive envelope is assumed to be equal to the negative envelope (Lowes and Altoontash 2003). Therefore, instead of 16 parameters as shown in **Fig. 6.4**, 8 parameters (4 key points) are sufficient to define the envelope response of the shear panel. **Fig. 6.5** shows an example of the shear stress-strain envelope of the joint specimen tested by Shin and Lafave (Shin and Lafave 2004), where the response envelope can be simplified as four linear segments connecting four key points ($P1$ through $P4$). The initial point $P1$ is taken as joint shear cracking, whereas reinforcement yielding occurs at $P2$, and $P3$ is defined as joint shear strength. After $P3$, the shear stress-strain curve becomes flat or decreases slightly in slope. Thus Kim's model (Kim and LaFave 2009; LaFave and Kim 2011) has considered the same pattern of four linear segments as assumed in *pinching4* material (**Fig. 6.4**).

Fig.6.5: Shear stress-strain response envelope (Shin and Lavafe 2004)

Kim and LaFave (2009) proposed a simple and unified joint shear behaviour model, in which the relationships between the four key points $P1$ to $P4$ can be expressed as

as the values in **Table 6.1**. Joint shear strength (v_j) and joint shear strain (γ_j) are evaluated through Eqs. (6-3) and (6-4), respectively.

$$v_j \text{ (Mpa)} = \alpha_t \beta_t \eta_t \lambda_t (JI)^{0.15} (BI)^{0.30} (f_c')^{0.75} \quad (6-3)$$

$$\gamma_j \text{ (rad)} = \alpha_{\gamma t} \beta_{\gamma t} \eta_{\gamma t} \lambda_{\gamma t} BI (JI)^{0.10} \left[\frac{v_j}{f_c'} \right]^{-1.75} \quad (6-4)$$

where α_t indicates in-plane geometry (taken as 1.0 for interior joints, 0.70 for exterior joints and 0.5 for knee joints); β_t indicates out-of-plane geometry (taken as 1.0 for 0 or 1 transverse beam, 1.2 for 2 transverse beams); η_t is joint eccentricity ($\eta_t = (1 - e/b_c)^{0.67}$, taken as 1.0 with no eccentricity); λ_t is taken as 1.31 (by default); f_c' is concrete compressive strength; JI is joint transverse reinforcement index (defined as $(\rho_j \cdot f_{y j})/f_c'$ where ρ_j is the volumetric joint transverse reinforcement ratio and $f_{y j}$ is the yield stress of the joint transverse reinforcement); BI is the beam reinforcement index (defined as $(\rho_b \cdot f_{y b})/f_c'$ where ρ_b is the beam reinforcement ratio and $f_{y b}$ is the yield strength of the beam reinforcement); $\alpha_{\gamma t}$ indicates in-plane geometry ($\alpha_{\gamma t} = (JPRU)^{2.10}$, where $JPRU$ is in-plane geometry parameter; which $JPRU = 1.0$ for interior joint when A_{sh} ratio is greater or above 0.7 and $JPRU = 1.0/1.2$ when A_{sh} ratio is below 0.7); A_{sh} ratio is the provided to recommended amount of joint transverse reinforcement according to ACI-ASCE (2002); $\beta_{\gamma t}$ indicates out-of-plane geometry (taken as 1.0 for 0 or 1 transverse beam, 1.4 for 2 transverse beams); $\eta_{\gamma t}$ is joint eccentricity ($\eta_{\gamma t} = (1 - e/b_c)^{-0.60}$, taken as 1.0 with no eccentricity); $\lambda_{\gamma t}$ is taken as 0.00549 (by default). For RC beam-column sub-assemblages with no joint transverse reinforcement, a virtual JI of 0.0139 is needed in Eq. (6-3) to predict the joint shear strength. Using the same virtual JI , the value of $\lambda_{\gamma t}$ is taken as 0.00761 instead of 0.00549 in Eq. (6-4) for the joint shear deformation prediction.

Table 6. 1: Unified joint shear behavior model (Kim and LaFave 2009)

Point	Shear Stress, v	Shear Strain, γ
$P1$	$0.442 v_j$	$0.0197 \gamma_j$
$P2$	$0.890 v_j$	$0.362 \gamma_j$
$P3$	v_j	γ_j
$P4$	$0.900 v_j$	$2.02 \gamma_j$

After obtaining joint shear strength and shear strain at $P3$ (v_j, γ_j) by Eqs. (6-3) and (6-4), shear stress and strain values at $P1$, $P2$ and $P4$ are calculated according to the relationships in **Table 6.1**. Shear deformation at $P4$ reaches approximately twice that of the joint shear strain (γ_j). This value typically ranges from 0.03 to 0.05 radians (Shin and Lafave 2004). After obtaining the shear stress and strain at four key points ($P1$ through $P4$), the envelope curve can be defined for both positive and negative regions in OpenSees.

6.3.2.1.2 Unload-reload path

The pinching and the shape of hysteresis curve is greatly affected by parameters that define unload-reload path as shown in **Fig 6.4**. **Table 6.2** shows the parameters defining unload-reload path, and the corresponding descriptions.

Table 6.2: Parameters for unload-reload path for shear panel in OpenSees

Parameter	Descriptions
$rDispP$	Ratio of the deformation at which reloading occurs to the maximum hysteretic deformation demand.
$rDispN$	Ratio of the deformation at which reloading occurs to the minimum hysteretic deformation demand.
$rForceP$	Ratio of the force at which reloading begins to the force corresponding to the maximum hysteretic deformation demand.

$rForceN$	Ratio of the force at which reloading begins to the force corresponding to the minimum hysteretic deformation demand.
$uForceP$	Ratio of strength developed upon unloading from negative load to the maximum strength developed under monotonic loading.
$uForceN$	Ratio of strength developed upon unloading from negative load to the minimum strength developed under monotonic loading.

6.3.2.1.3 Hysteretic damage rules

Hysteretic damage of the *pinching4* material model is simulated through deterioration in unloading stiffness degradation, reloading stiffness degradation and strength degradation (Lowe and Altoontash 2003) as shown in **Fig. 6.6**.

(a) Unloading stiffness degradation damage mode

(b) Reloading stiffness degradation damage mode

(c) Strength degradation damage mode

Fig.6.6: Hysteresis damage modes (Lowe and Altoontash 2003)

A general damage index is given by Park and Ang (1985) as follows:

$$\delta_i = \left[\alpha_1 (d_{max})^{\alpha_3} + \alpha_2 \left(\frac{E_i}{E_{monotonic}} \right)^{\alpha_4} \right] \leq \text{limit} \quad (6-5)$$

$$d_{max} = \text{maximum} \left[\frac{(d_{max})_i}{d_{efmax}}, \frac{(d_{min})_i}{d_{efmin}} \right] \quad (6-6)$$

It is notable that i refers to the current displacement increment; δ_i is the damage index ($\delta_i = 0$ represents no damage and $\delta_i = 1$ means the case of maximum damage); $\alpha_1, \alpha_2, \alpha_3$ and α_4 are parameters for use in fitting the damage rule to experimental data; E_i is the accumulated hysteretic energy ($E_i = \int_{load\ history} dE$); $E_{monotonic}$ is the energy required to achieve the deformation that defines failure;

$d_{ef_{max}}$ and $d_{ef_{min}}$ are the positive and negative deformations that define failure; and $(d_{mx})_i$ and $(d_{mx})_o$ are the maximum and the minimum hysteretic deformation demands.

For the case of unloading stiffness degradation (**Fig. 6.6(a)**), current unloading stiffness k_i can be evaluated via :

$$k_i = k_o(1 - \delta k_i) \quad (6-7)$$

where k_o is the initial unloading stiffness and δk_i is the current value of the stiffness damage index estimated from Eq. 6-5. Envelope strength degradation (**Fig. 6.6(c)**) is simulated in the same way as:

$$(f_{max})_i = (f_{max})_o (1 - \delta f_i) \quad (6-8)$$

where $(f_{max})_i$ is the current envelope maximum strength; $(f_{max})_o$ is the initial envelope maximum strength; and δf_i is the current value of the strength damage index estimated from Eq. 6-5.

The reduction in strength that is observed upon reloading is simulated by applying the damage rule to define an increase in the maximum hysteretic deformation (**Fig. 6.6(b)**) as:

$$(d_{max})_i = (d_{max})_o (1 + \delta d_i) \quad (6-9)$$

where $(d_{max})_i$ is the current deformation that defines the end of the reloading cycle for increasing deformation demand; $(d_{max})_o$ is the maximum hysteretic deformation demand and δd_i is the current value of the reloading-strength damage index estimated from Eq. 6-5. **Table 6.3** gives the corresponding terms of sixteen parameters defining the three damage rules of shear panel in OpenSees.

Table 6.3: Parameters defining the three damage rules of shear panel in OpenSees

Terms in OpenSees	Description
$gK_1, gK_2, gK_3, gK_4,$ gK_{Lim}	Values controlling cyclic degradation model for unloading stiffness degradation. Parameters $gK_1, gK_2, gK_3,$ and gK_4 are the values for $\alpha_1, \alpha_2, \alpha_3$ and α_4 in Eq. (6-5) for δk_i . Parameter gK_{Lim} is the damage index limit for stiffness.
$gD_1, gD_2, gD_3, gD_4,$ gD_{Lim}	Values controlling cyclic degradation model for reloading stiffness degradation. Parameter gD_1, gD_2, gD_3 and gD_4 are the values for $\alpha_1, \alpha_2, \alpha_3$ and α_4 in Eq. (6-5) for δd_i . Parameter gD_{Lim} is the damage index limit for displacement.
gF_1, gF_2, gF_3, gF_4 gF_{Lim}	Values controlling cyclic degradation model for strength degradation. Parameter gF_1, gF_2, gF_3 and gF_4 are the values for $\alpha_1, \alpha_2, \alpha_3$ and α_4 in Eq. (6-5) for δf_i . Parameter gF_{Lim} is the damage index limit for force.
gE	Used to define maximum energy dissipation under cyclic loading. Total energy dissipation capacity is defined as this factor multiplied by the energy dissipated under monotonic loading.

6.3.2.2 Constitutive Model for Bar-slip

Lowes and Altoontash (2003) proposed a calibration model for the bar-slip springs by using data from experimental studies on anchorage-zone specimens with some assumptions of the bond stress distribution within the joint, as follows.

- (i) The bond stress along the anchorage length of the reinforcing bar is assumed to be uniform for reinforcement that remains elastic or piecewise uniform for reinforcement loaded beyond yield (**Fig. 6.7**).
- (ii) Slip is assumed to define the relative movement of the reinforcement bar with respect to the perimeter of the joint and is a function of the strain distribution along the bar.
- (iii) Bar is assumed to exhibit zero slip at the point of zero bar stress.
- (iv) Based on studies by Eligehausen et al. (1983) and Lowes (1999), it was recommended by Lowes and Altoontash (2003) that bond strength deteriorates once slip exceeds 3 mm and the post peak stiffness is defined as equal to 10% of the initial stiffness. The slip value agrees with the measured values from pull-out test (Section 5.2.4) as shown in **Fig 5.4**.

Fig. 6.7: Bond stress and bar stress distribution of a bar anchored in a beam-column joint (Lowes and Altoontash 2003)

With the above assumptions, bar stress-slip relationship is defined as follows (Lowes and Altoontash 2003):

$$slip = \int_0^{l_{fs}} \tau_E \frac{\pi d_b}{A_b} \cdot \frac{1}{E_s} x dx \quad (6-10)$$

$$\begin{aligned}
&= 2 \frac{\tau_E l_{fs}^2}{E_s d_b} \text{ for } f_s < f_y \\
slip &= \int_0^{l_e} \frac{4}{d_b} \frac{\tau_E}{E_s} x dx + \int_{l_e}^{l_y+l_e} \left(\frac{f_y}{E_s} + \tau_y \frac{4}{d_b} \frac{(x-l_e)}{E_h} \right) dx \\
&= 2 \frac{\tau_E l_e^2}{E_s d_b} + \frac{f_y}{E_s} l_y + 2 \frac{\tau_y l_y^2}{E_h d_b} \text{ for } f_s \geq f_y
\end{aligned} \tag{6-11}$$

where f_s is the bar stress at the joint perimeter; f_y is the yield strength of steel reinforcement; E_s is the elastic modulus of reinforcing steel bar; E_h is the hardening modulus of steel bar assuming the steel bar response to be represented by a bilinear curve; τ_E is the bond strength for elastic steel bar; τ_y is the bond strength for yielded steel bar; A_b is the nominal steel bar area; d_b is the nominal steel bar diameter; l_{fs} is the length of steel bar at a particular slip ($l_{fs} = \frac{f_s}{\tau_{ET}} \cdot \frac{A_b}{\pi d_b}$); l_e and l_y define respectively the length of the steel bar for which bar stress is less than or greater than the yield strength ($l_e = \frac{f_y}{\tau_{ET}} \cdot \frac{A_b}{\pi d_b}$; $l_y = \frac{f_s - f_y}{\tau_{YT}} \cdot \frac{A_b}{\pi d_b}$). According to Lowes and Altoontash (2003), for the case of $l_e + l_y$ greater than the width of the joint, the deterioration of bond strength under cyclic loading is more severe and it is appropriate to assume reduced bond strength in the elastic region of the reinforcing bar. Hence, this bar-slip model has considered the case of insufficient anchorage length which may occur in the interior beam-column sub-assemblages (CIJ-S-1 and EIJ-S-0) with $h_c = 19.23 d_b$, which was slightly less than $20 d_b$ as required by joint ACI-ASCE (ACI-ASCE 2002). It is noted that h_c is the column depth and d_b is the diameter of beam bars passing through joints.

Average bond strength values are of importance to establishing bar stress-slip relationships. In determining the average bond stress at elastic stage, Lowes employed the bond-slip model by (Eligehausen et al. 1983), which assumes zero to maximum bond strength developed at the two ends of an embedded steel bar. This results in an average bond strength equal to 71% of the maximum bond strength (Eligehausen et al. 1983). From the pull-out tests as discussed in Section 5.2.4.1, the maximum bond stress at elastic stage for steel reinforcement embedded in concrete

and ECC was respectively $3.0\sqrt{f_c'}$ and $3.4\sqrt{f_c'}$, when steel reinforcement was subjected to tensile load (pull-out load). Therefore, the calculated average bond stress was $2.13\sqrt{f_c'}$ and $2.41\sqrt{f_c'}$ for steel reinforcement embedded in concrete and ECC, respectively, if 71% of the maximum bond strength was considered. However, average bond stress is taken as $1.8\sqrt{f_c'}$ (i.e. 71% of $2.5\sqrt{f_c'}$) for steel reinforcement embedded in concrete due to greater number of pull-out tests conducted by Eligehausen et al. (1983). Meanwhile, average bond stress for steel reinforcement embedded in ECC was taken as $2.05\sqrt{f_c'}$ based on 14% of enhancement by ECC as demonstrated in pull-out tests as described in Section 5.2.4.1. When the bar has yielded in tension, the average bond strength is taken as $0.4\sqrt{f_c'}$ based on (Shima et al. 1987) for concrete joints. For ECC joints, the average bond strength at post-yield is assumed as in the same trend (i.e. 14% enhancement) as in elastic stage. This assumption is acceptable, as the proposed bond-slip model in post-yield state of embedded reinforcement in **Table 5.5** also has almost the same enhancement factor (i.e. 12.5% enhancement) for ECC compared to concrete.

For steel reinforcement in compression anchorage zone, data provided by previous research studies (Eligehausen et al. 1983; Lowes 1999; Shima et al. 1987; Viwathanatepa et al. 1979) were used to establish the average bond strength for steel reinforcement embedded in concrete (see **Table 6.4**). According to Yu (2012), the average bond strength in compression (τ_{EC}) during elastic stage becomes larger than that in tension (τ_{ET}) due to Poisson effect which causes the diameter of a reinforcing bar under compression increases. Thus, $2.2\sqrt{f_c'}$ is used for compression (Lowes 1999) but for tension it is $1.8\sqrt{f_c'}$. When a bar has yielded, the Poisson effect is more dominant, leading to the average bond strength (τ_{YC}) much greater than that in elastic compression (τ_{EC}) as indicated in **Table 6.4**.

For steel reinforcement embedded in ECC, it is further assumed that average bond strength for compression in elastic and inelastic stages are factored by 1.14 as shown in **Table 6.4** since the author did not conduct anchorage bond test subjected to compression. These values are assumed due to the lack of experimental data for

anchorage bond of reinforcement embedded in ECC subjected to compression. However, observations from test results in Section 4.2.2.4 showed that compressive bar stress was very small compared to tensile bar stress in the reinforcement passing through the joint and yielding of rebar in compression did not occur. Therefore, it is unlikely that at post-yield stage the bond stress exceeds the average bond strength in compression.

Table 6.4: Average bond strength for bar-slip

Bar stress, f_s	Average bond strength in concrete joints (MPa)	Average bond strength in ECC joints (MPa)
Tension, $f_s < f_y$	$\tau_{ET} = 1.8\sqrt{f_c'}$	$\tau_{ET} = 2.05\sqrt{f_c'}$
Tension, $f_s > f_y$	$\tau_{YT} = 0.4\sqrt{f_c'}$	$\tau_{YT} = 0.46\sqrt{f_c'}$
Compression, $-f_s < f_y$	$\tau_{EC} = 2.2\sqrt{f_c'}$	$\tau_{EC} = 2.50\sqrt{f_c'}$
Compression, $-f_s > f_y$	$\tau_{YC} = 3.6\sqrt{f_c'}$	$\tau_{YC} = 4.10\sqrt{f_c'}$

Due to the limitation of user interface in OpenSees, the average bond strength proportional to $\sqrt{f_c'}$ is predefined in which the user was unable to modify the average bond strength value in the bar-slip material in OpenSees. Therefore the enhanced average bond strength (i.e. 114%) for ECC joints was incorporated through equivalent compressive strength of concrete, f_{ce}' in the bar-slip material. The calculation in obtaining f_{ce}' is shown in **Table 6.5** for the case of interior beam-column sub-assemblages with ECC (EIJ-NS-0 and EIJ-S-0). As can be seen from **Table 6.5**, the equivalent compressive strength of concrete, $f_{ce}' = 65.25 \text{ MPa}$ was employed (instead $f_c' = 50.3 \text{ MPa}$) to represent enhanced bond strength by ECC in bar-slip material.

Table 6.5: Calculation of equivalent compressive strength of concrete f_{ce}' in ECC joints

Step	Calculation
1. Identify f_c' for ECC joints	$f_c' = 50.3 \text{ MPa}$
2. Calculate the average bond strength for elastic tensile stress of rebar $\tau_{ET} = 2.05\sqrt{f_c'}$	$\tau_{ET} = 14.54 \text{ MPa}$
3. Back calculate the equivalent compressive strength of concrete, f_{ce}' based on $1.8\sqrt{f_c'}$ since this is pre-defined in the bar-slip model.	$\sqrt{f_{ce}'} = 14.54/1.8$ $f_{ce}' = 65.25 \text{ MPa}$

Relationship between bar stress and spring force is of great importance to the development of bar-slip spring constitutive model. In a beam-column joint model, the axial and flexural forces from the beam and column members are transferred to the joint core through tension and compression forces in the bar-slip springs. For the case of tensile spring force, it is appropriate to assume that all of the spring force is carried by the reinforcing steel and transferred into the joint through bond. However, for compressive loading, total compressive force is carried by the concrete and the reinforcing steel in compression (Lowes and Altoontash 2003). Therefore, only a fraction of the total compression is transferred into the joint through bond stress. The concrete compression force and the steel compression force are computed based on the recommendations of ACI Committee 318 (Committee et al. 1999). More details of the compressive spring force can be found in Lowes and Altoontash (2003).

6.4 Calibration of Cyclic Response for Shear Panel

Due to the limited test data for calibrating the unload-reload path and three damage rules for ECC shear panel, those parameters as mentioned in Section 6.3.2.1.2 and 6.3.2.1.3 were input in OpenSees based on Lowes's study (Lowes et al. 2003) in which experimental data provided by Stevens et al. (1991) were used. However, the analysis could not be continued by using the first trial set of parameters. After several trials had been made, the effects for each parameter were identified. For the same series of simulations, the variations of input parameters for defining cyclic response were kept to the minimal. Finally, parameters in defining unload-reload path and three damage rules are determined as in **Table 6.6**. It is notable that the damage index in Eq. 6.5 was in two terms i.e. a function of displacement history and a energy accumulation. User can select damage index in a function of displacement only or combine of two (Mazzoni et al. 2006). Thus in this study, only damage index in displacement term was used for the sake of reducing uncertain parameters. Therefore the values of $gK_2, gK_4, gD_2, gD_4, gF_2$ and gE are zeros as shown in **Table 6.6**.

Table 6.6: Input parameters defining cyclic response of shear panel

Parameter	Interior beam-column sub-assemblages				Exterior beam-column sub-assemblages			
	CIJ-NS-0	EIJ-NS-0	CIJ-S-1	EIJ-S-0	CEJ-A-1	EEJ-A-0	EEJ-A-1	EEJ-NA-1
$rDispP$ $rDispN$	0.25	0.25	0.25	0.25	0.50	0.25	0.25	0.25
$rForceP$ $rForceN$	0.15	0.15	0.15	0.15	0.35	0.30	0.30	0.30
$uForceP$ $uForceN$	-0.20	-0.15	-0.20	-0.20	-0.10	-0.10	-0.10	-0.10
$gK_1, gK_2,$ gK_3, gK_4 gK_{Lim}	1.13, 0.0, 0.1, 0.0, 0.90	1.13, 0.0, 0.1, 0.0, 0.90	1.13, 0.0, 0.1, 0.0, 0.90	1.13, 0.0, 0.1, 0.0, 0.90	1.13, 0.0, 0.1, 0.0, 0.75	1.13, 0.0, 0.1, 0.0, 0.50	1.13, 0.0, 0.1, 0.0, 0.20	1.13, 0.0, 0.1, 0.0, 0.20
$gD_1, gD_2,$ gD_3, gD_4 gD_{Lim}	0.12, 0.0, 0.23, 0.0, 0.90	0.12, 0.0, 0.23, 0.0, 0.90	0.12, 0.0, 0.23, 0.0, 0.90	0.12, 0.0, 0.23, 0.0, 0.90	0.12, 0.0, 0.23, 0.0, 0.50	0.12, 0.0, 0.23, 0.0, 0.50	0.12, 0.0, 0.23, 0.0, 0.80	0.12, 0.0, 0.23, 0.0, 0.80

$gF_1, gF_2,$	1.11, 0.0,	1.11, 0.0,	1.11, 0.0,	1.11, 0.0,	1.11, 0.0,	1.11, 0.0,	1.11, 0.0,	1.11, 0.0,
gF_3, gF_4	0.32, 0.0,	0.32, 0.0,	0.32, 0.0,	0.32, 0.0,	0.32, 0.0,	0.32, 0.0,	0.32, 0.0,	0.32, 0.0,
gF_{Lim}	0.35	0.62	0.35	0.50	0.65	0.65	0.70	0.75
gE	0	0	0	0	0	0	0	0

6.5 Results of Simulation

6.5.1 Interior beam-column sub-assemblages (IJ series)

Numerical results on hysteresis response of the four interior beam-column sub-assemblages are shown in **Fig. 6.8**. **Table 6.7** lists the maximum load capacity and the corresponding drift ratio. The mean ratio of the numerical to experimental load capacities is 1.14, with a coefficient of variation of 0.078. The corresponding drift ratios for numerical load capacities were compared with drift ratios observed during the tests. The mean ratio of the corresponding drift ratio (numerical to experimental) is 0.97 as shown in **Table 6.7**.

As can be seen, the simulated envelopes are generally greater than the test results. This could be due to over estimation of the shear panel response by using Kim's model (Kim and LaFave 2009; LaFave and Kim 2011), and too simplification of the bar-slip model. Simulated responses are symmetrical in positive and negative loading cycles due to the assumption that positive and negative shear panel envelopes are identical. The simulated stiffness values are far greater than the actual stiffness values, possibly due to inevitable gaps in the test set-up which could not exactly emulate ideal boundary conditions .

Table 6.7: Comparison of numerical and experimental results (IJ series)

Specimen	Numerical		Experimental		$\frac{V_n}{V_e}$	$\frac{DR_n}{DR_e}$
	Load capacity V_n (kN)	Corresponding DR_n (%)	Load capacity V_e (kN)	Corresponding DR_e (%)		
CIJ-NS-0	27.20	3.5	26.18	3.5	1.03	1.00
EIJ-NS-0	26.76	2.0	23.82	3.0	1.12	0.67
CIJ-S-1	51.32	3.0	40.98	3.5	1.25	0.86
EIJ-S-0	54.31	5.5	47.38	4.0	1.15	1.38
Mean value					1.14	0.97
Coefficient of variation					0.078	0.31

For specimens with ECC joints (EIJ-NS-0 and EIJ-S-0), the assumption of relatively high stiffness for interface-shear spring may contribute to over prediction of overall response envelopes as well as stiffness of the sub-assemblages. Widening of cracks at vertical joint interfaces was observed in ECC specimens during the test. Thus, an appropriate material model for the interface-shear spring needs to be established for beam-column joints with ECC material. It should be noted that for EIJ-NS-0, pinching of hysteresis loop due to rebar fracture could not be simulated due to limitations of the bar-slip component.

Fig. 6.8: Comparison of simulated response with test result (IJ series)

Fig. 6.9 shows the force-slip relations of the top bar-slip component at the right beam. It was observed that for both non-seismic and seismic design specimens, the bar-slip component in ECC specimens (EIJ-NS-0 and EIJ-S-0) demonstrated reduction in slips by around 42-44% of that obtained in concrete specimens (CIJ-NS-0 and CIJ-S-1). The attainment of maximum forces were identical under the same design of specimens as shown in **Fig. 6.9**. Similar trend can be seen when the top bar-spring is in compression under load reversal. A reduction of slip value was observed in ECC specimens when compared to the corresponding concrete specimens. This observation is due to the enhanced bond strength of ECC (see **Table 6.4**) in the bar-slip component in OpenSees.

**Fig. 6.9: Force-slip for bar-slip component located at the right beam
(top) for IJ series**

6.5.2 Exterior beam-column sub-assemblages (EJ series)

Fig. 6.10 shows the comparison of numerical predictions with test results for exterior joint series. Fairly good agreement is obtained between them in terms of the load capacity and global response. **Table 6.8** lists the load capacity and corresponding drift ratio. The mean ratio of numerical to experimental load capacities is 1.11 with a coefficient of variation of 0.099. Simulated corresponding drift ratios for all specimens were smaller than experimental results, with a mean ratio of 0.63 as shown in **Table 6.8**. Similar to IJ series, there was a gap of initial stiffness between simulated and experimental responses as shown in **Fig. 6.10**.

Table 6.8: Comparison of numerical and experimental results (EJ series)

Specimen	Numerical		Experimental		$\frac{V_n}{V_e}$	$\frac{DR_n}{DR_e}$
	Load capacity V_n (kN)	Corresponding DR_n (%)	Load capacity V_e (kN)	Corresponding DR_e (%)		
CEJ-A-1	27.65	2.0	26.09	3.5	1.06	0.57
EEJ-A-0	27.32	1.5	22.72	3.0	1.20	0.50
EEJ-A-1	27.52	2.5	23.06	3.0	1.19	0.83
EEJ-NA-1	22.74	2.5	23.29	4.0	0.98	0.63
Mean value					1.11	0.63
Coefficient of variation					0.099	0.23

Fig. 6.10: Comparison of simulated response with test result (EJ series)

Fig. 6.11 shows the force-slip relations of the bar-slip component at the right beam (top) for EJ series. Generally, ECC joints (EEJ-A-0, EEJ-A-1 and EEJ-NA-1) showed a reduction of slip (25-36%) in bar-slip component for both tension and compression as compared to concrete joint (CEJ-A-1).

Fig. 6.11: Force-slip for bar-slip component located at the right beam (top) for EJ series

6.6 Summary

Simulated results indicate that the behaviour of beam-column sub-assemblages can be modelled by using OpenSees if proper joint shear model and bond-slip behaviour of reinforcement are employed. More importantly, the set of values in **Table 6.6** have to be adjusted to obtain the “closest” envelopes. In this study, the enhanced bond strength by ECC material was considered in the bond-slip behaviour. But due to simplification of the bar-slip model and the assumptions made (Section 6.3.2.2), slip values obtained from OpenSees simulations were well below 3.0 mm, in which

bond deterioration did not commence. Therefore, pinching of hysteresis response due to bar-slip could not be simulated at all.

The use of Kim's model in predicting joint shear panel response for ECC joints may over estimate the joint shear strength and under estimate the joint shear strain, leading to inaccuracy of the simulated response. Compared to IJ series, interface cracks did not occur at the perimeter of the joint core for EJ series during the test. Therefore, the assumption of relatively stiff interface-shear spring is acceptable for EJ series. Thus, the simulated response in EJ series was closer to test results compared to IJ series.

In order to simulate the cyclic response with greater accuracy, a proper hysteretic damage model which can simulate deterioration in unloading stiffness degradation, reloading stiffness degradation and strength degradation is needed for shear panel in the beam-column joint model. An appropriate calibration method need to be established for shear panel with ECC, so that the actual behaviour of the joints can be better simulated.

CHAPTER 7: CONCLUSIONS AND FUTURE WORK

7.1 Conclusions

This research investigates the seismic behaviour of RC beam-column sub-assemblages with ECC in the joint cores under lateral cyclic loading. The effect of ECC in enhancing structural performance in both interior and exterior beam-column sub-assemblages was evaluated through two series of experimental tests. Besides, a series of component pull-out tests on deformed bars embedded in concrete or ECC was conducted. An analytical bond-slip model was proposed for predicting the force-slip relationship of reinforcement anchored in concrete or ECC. Finally, component-based joint modelling was carried out to simulate hysteresis response of the beam-column sub-assemblages. The major innovations of this study are the utilisation of minimum amount of local tailored ECC material in the joint cores by means of enhancing seismic performance of RC beam-column sub-assemblages, the development of analytical model for reinforcement anchored in ECC and the bar-slip material model which has considered the effect of ECC.

Seismic behaviour of interior beam-column sub-assemblages

According to strong-column-weak-beam design concept in beam-column joints, columns have to be designed to have a certain amount of flexural strength compared to beams to avoid brittle joint shear failure mode. In order to study the effect of ECC in joint cores, strong-column-weak-beam requirement was not imposed on interior joints, to allow failures in the joints of the specimens. In addition, column axial load was not applied to interior joints to avoid any enhancement in joint shear strength. From the test results, joint shear failure was observed in both concrete specimens, but not in ECC specimens. The use of ECC joints has significantly changed brittle joint shear failure mode to ductile beam flexural mode due to its superior shear strain capacity. For beam-column sub-assemblages with non-seismic design (EIJ-NS-0), ECC improved total energy dissipation by around 76% although the load capacity was slightly less compared with that of the corresponding concrete specimen (CIJ-NS-0). On the other hand, nearly the same total energy dissipation

was attained for beam-column sub-assemblages with seismic design (CIJ-S-1 and EIJ-S-0), but ECC slightly improved the load capacity (EIJ-S-0) by around 15% compared to concrete specimen (CIJ-S-1) in spite of total elimination of stirrups from the joint core. ECC exhibited multiple cracking and damage tolerance behaviour. Besides, ECC was capable of reducing shear distortion of joint cores by around 85-90% during a drift ratio of 6.0% compared to concrete joint cores. Compatible deformation between longitudinal bars and ECC in the joint core was observed at the initial stage compared to concrete joints. However, bond deterioration was inevitable with widening of cracks at the joint interfaces as drift ratio increased. Overall, the feasibility of using ECC without stirrups in the joints is clearly demonstrated in this series of tests. Thus, using ECC materials can avoid reinforcement congestion and construction difficulties at the joint cores.

Seismic behavior of exterior beam column sub-assemblages

In this series of test, all exterior beam-column sub-assemblages were designed and detailed according to Eurocode 8 complying with the requirement of strong-column-weak-beam approach. Therefore, the effectiveness of ECC in exterior joint cores was investigated under the condition of beam-column sub-assemblages subject to beam failure mode. The effects of joint stirrups and column axial load were also studied. Experimental results indicated that the use of ECC in joint cores did not improve the load capacity of exterior beam-column sub-assemblages compared to concrete specimens under the beam failure mode. Hysteresis response was controlled by flexural strength of the beams and columns, rather than the joint shear strength. Due to unforeseen reason, the control specimen (CEJ-A-1) had to be re-constructed. Therefore, it had different material properties of concrete and steel reinforcement from the other ECC specimens (EEJ-A-0, EEJ-A-1 and EEJ-NA-1). Due to its relatively higher compressive strength of concrete, its load capacity was about 20% greater than those of ECC specimens. However, a different failure mode was detected in concrete specimen (CEJ-A-1), viz. crushing of concrete cover at the far end of column could be seen during the late loading stage. ECC specimens did not show this failure, as ECC could provide lateral resistance against cover crushing and spalling.

For exterior beam-column sub-assemblages, from the calculated joint shear stress values, the contribution of ECC in joints is rather limited as the demand on joint shear stress was low. Better bond behaviour was observed in ECC specimens because bar-slippage did not commence prior to beam failure in those specimens. Due to low shear demand, ECC joint without stirrups (EEJ-A-0) performed closely to ECC joint with stirrups (EEJ-A-1) in terms of load-carrying capacity, energy dissipation, joint shear stress and anchorage bond. The effect of column axial load can be more clearly observed in the behaviour of joint shear. Joint shear demand was reduced by 16% for ECC joint with column axial load (EEJ-A-1) compared to ECC joint without column axial load (EEJ-NA-1).

Bond-slip behaviour of reinforcement embedded in ECC

Bond-slip behaviour of short (five times rebar diameter) and long (twenty times rebar diameter) embedment lengths of steel reinforcement in well-confined concrete and ECC was studied through pull-out tests. Under pull-out failure at elastic stage, the maximum local bond stress for short reinforcement in ECC cores was 14% greater than that in concrete cores. On the other hand, beyond the yield capacity, long reinforcement experienced softening and eventually fractured under pull-out loads. Force-slip relationships indicated that long reinforcement embedded in ECC developed nearly the same yield, ultimate and rupture forces with concrete specimens. However, slip values at the loaded end were considerably smaller than those in concrete specimens.

Analytical bond-slip model for predicting force-slip relationships

An analytical bond-slip model was proposed based on the elastic bond-stress and post-yield bond stress values obtained from the pull-out tests. This model can give good predictions of force-slip relationships for long reinforcement in concrete and ECC. Besides, a parametric study is conducted on the effect of embedment length in concrete and ECC for four different types of failure mode.

Component-based joint model in simulating seismic response of beam-column sub-assemblages

Component-based joint model (*beam-column joint element*) proposed by (Lowes and Altoontash 2003) was used to simulate seismic response of the tested beam-

column sub-assemblages through OpenSees. This element comprised three major components, viz. shear panel, bar-slip springs and interface shear springs. Enhanced bond strength of ECC was incorporated in the bar-slip component of the *beam-column joint element*. Shear panel envelopes were simulated through the use of joint shear model proposed by (Kim and LaFave 2009). Simulated results indicated that the response envelopes of beam-column sub-assemblages can be modelled by using OpenSees if appropriate joint shear model and bond-slip model are employed. Pinching of hysteresis response was poorly simulated due to limited data for calibration of unload-reload paths, hysteretic damage of the shear panel and bar-slip component for ECC joints. Force-slip relationships of bar spring indicate that slip values in ECC joints were generally lower than that in concrete joints. However, bond deterioration could not be simulated due to simplicity of the bar-slip model. The use of Kim's model in predicting joint shear panel response for ECC joints led to an overestimation of the joint shear strength and an underestimation of the joint shear strain, leading to inaccuracy of the simulated response. Therefore, an appropriate joint model which can represent ECC shear behaviour is needed to simulate shear panel response accurately. Similarly, a proper hysteretic damage model which can simulate unloading stiffness degradation, reloading stiffness degradation and strength degradation, is required for the ECC shear panel.

7.2 Future Works

This research demonstrates the possibility of using ECC in RC beam-column joints to mitigate brittle joint shear failure of RC moment resisting frames. However, the outputs of this research are insufficient to provide a comprehensive understanding of the behaviours of beam-column sub-assemblages with ECC catering all kind of design scenarios. Therefore, more investigations are needed and the following recommendations for future work are proposed:

- i) In the interior beam-column sub-assemblage tests, axial load was not applied to the columns. Therefore, it is of particular interest to quantify the degree of enhancement if the columns are subject to different levels of axial loads in future studies.

- ii) For specimens satisfying seismic design, the ratio of beam longitudinal bars to column reinforcement plays an important role in joint shear demand. If the ratio of beam longitudinal bars is smaller, joint shear demand is subsequently reduced. Therefore, these parameters can be considered in future work to study the effect of ECC for different levels of joint shear demand and flexural strength ratios. It is preferred that both series of tests maintain the same set of parameters so that structural behaviours can be compared.
- iii) To study the seismic behaviours of precast beam-column sub-assemblages with ECC in the joint core since precast structural system is commonly practiced in Singapore. Interface shear behaviour between precast elements and cast in place ECC can be investigated.
- iv) To conduct three-dimensional experimental tests in which transverse beam in the out of plane direction can be added to current beam-column sub-assembly for the purpose of evaluating shear strength of different configurations of beam-column connections. The effect of slab can be incorporated in this set-up to emulate the actual scenario.
- v) For component-based joint modelling, there are still lack of test data on the ECC shear panel and the interface-shear springs. Thus, some of these component tests have to be carefully planned and conducted in the future so that the measured values can be used for simulating the behaviours of sub-assemblages.

REFERENCES

- ACI-ASCE, J. (2002). "Recommendations for design of beam-column joints in monolithic reinforced concrete structures." ACI352R-02, American Concrete Institute, Farmington Hills, MI.
- Asano, K., and Kanakubo, T. (2012). "Study on size effect in bond splitting behavior of ECC." *High Performance Fiber Reinforced Cement Composites* 6, Springer, 137-144.
- Asano, K., and Kanakubo, T. (2012). "Study on Size Effect in Bond Splitting Behavior of ECC." *High Performance Fiber Reinforced Cement Composites* 6, 2(137-144).
- Ashtiani, M. S., Dhakal, R. P., and Scott, A. N. (2014). "Seismic performance of high-strength self-compacting concrete in reinforced concrete beam-column joints." *Journal of Structural Engineering*, 140(5), 04014002.
- Bandelt, M. J., and Billington, S. L. (2014). "Bond behavior of steel reinforcement in high-performance fiber-reinforced cementitious composite flexural members." *Materials and Structures*, 49(1), 71-86.
- Bandelt, M. J., and Billington, S. L. (2014). "Bond behaviour of steel reinforcement in high-performance fiber-reinforced cementitious composite flexural members." *Materials and Structures*, 1-6.
- Bigaj, A. J. (1995). "Bond Behaviour of Deformed Bars in NSC and HSC-Experimental Study." *Report No. 25.5-95-11*, Steven Laboratory, Faculty of Civil Engineering, Delft University of Technology.
- Billington, S., and Yoon, J. (2004). "Cyclic response of unbonded posttensioned precast columns with ductile fiber-reinforced concrete." *Journal of Bridge Engineering*, 9(4), 353-363.
- BSI (2004). "Eurocode 2: Design of Concrete Structures: Part 1-1: General Rules and Rules for Buildings." *BS EN 1992-1-1*, British Standards Institution, London.
- BSI (2004). "Eurocode 8: Design of structures for earthquake resistance." *BS EN 1998-1*, British Standards Institution, London.

- BSI (2005). "Eurocode 3: Design of steel structures-Part 1-8: Design of joints." *EN 1993-1-8:2005*, British Standards Institution, London.
- Chao, S.-H., Naaman, A. E., and Parra-Montesinos, G. J. (2010). "Local Bond Stress-Slip Models for Reinforcing Bars and Prestressing Strands in High-Performance Fiber-Reinforced Cement Composites." *ACI Special Publication*, 272, 151-172.
- Code, M. (2010). "International Federation for Structural Concrete (fib)." *Federal Institute of Technology Lausanne-EPFL, Section Génie Civil, Switzerland*.
- Committee, A., Institute, A. C., and Standardization, I. O. f. "Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary (ACI 318R-99)." American Concrete Institute.
- Corley, W. G. (1966). "Rotational capacity of reinforced concrete beams." *J. Struct. Div.*, 92(5), 121-146.
- Ehsani, M. R., and Wight, J. K. (1985). "Exterior Reinforced Concrete Beam-to-Column Connections Subjected to Earthquake-Type Loading." *Journal Proceedings*, 82(4).
- Eligehausen, R., Popov, E. P., and Bertero, V. V. (1983). "Local bond stress-slip relationships of deformed bars under generalized excitations: experimental results and analytical model."
- Eligehausen, R., Popov, E. P., and Betero, V. V. (1983). "Local Bond Stress-Slip Relationships of Deformed Bars under Generalized Excitations." *Report No. UCB/EERC-83/23*, College of Engineering, University of California, Berkeley.
- Engström, B., Magnusson, J., and Huang, Z. (1998). "Pull-Out Bond Behavior of Ribbed Bars in Normal and High-Strength Concrete with Various Confinements." In: *Leon, R. Editor. Bond and Development of reinforcement: A Tribute to Dr. Peter Gergely*, *ACI Special Publication*, 180(10), 215-242.
- Filippou, F. C., Popov, E. P., and Bertero, V. V. (1983). "Effects of bond deterioration on hysteretic behavior of reinforced concrete joints."

- Fischer, G., and Li, V. C. (2002). "Effect of matrix ductility on deformation behavior of steel-reinforced ECC flexural members under reversed cyclic loading conditions." *ACI Structural Journal*, 99(6).
- Fischer, G., and Li, V. C. (2002). "Effect of Matrix Ductility on Deformation Capacity Behavior of Steel-Reinforced ECC Flexural Members under Reversed Cyclic Loading Conditions." *ACI Structural Journal*, 99(6), 781-790.
- Fischer, G., and Li, V. C. (2002). "Influence of matrix ductility on tension-stiffening behavior of steel reinforced engineered cementitious composites (ECC)." *ACI Structural journal*, 99(1).
- Fischer, G., and Li, V. C. (2003). "Deformation Behavior of Fiber-Reinforced Polymer Reinforced Engineered Cementitious Composite (ECC) Flexural Members under Reversed Cyclic Loading Conditions " *ACI Structural Journal*, 100(1), 25-35.
- Fukuyama, H., Sato, Y., Li, V. C., Matsuzaki, Y., and Mihashi, H. (2000). "Ductile engineered cementitious composite elements for seismic structural applications." *CD Proceedings of the 12 WCEE, Paper*, 1672.
- Hakuto, S. (1995). "Retrofitting of Reinforced Concrete Moment Resisting Frames." PhD Thesis, University of Canterbury, Christchurch New Zealand.
- Hakuto, S., Park, R., and Tanaka, H. (1999). "Effect of deterioration of bond of beam bars passing through interior beam-column joints on flexural strength and ductility." *Structural Journal*, 96(5), 858-864.
- Hakuto, S., Park, R., and Tanaka, H. (2000). "Seismic load tests on interior and exterior beam-column joints with substandard reinforcing details." *Structural Journal*, 97(1), 11-25.
- Jaspart, J.-P. (2000). "General report: session on connections." *Journal of Constructional Steel Research*, 55(1), 69-89.
- JSCE (2008). "Recommendations for Design and Construction of High Performance Fiber Reinforced Cement Composites with Multiple Fine Cracks (HPFRCC)." *Concrete Engineering Series 82*, Japan Society of Civil Engineers.

- JSCE (2008). "Recommendations for design and construction of high performance fiber reinforced cement composites with multiple fine cracks (HPFRCC)." *Concrete engineering series no 82* Japan Society of Civil Engineers.
- Kalogeropoulos, G. I., Tsonos, A.-D. G., Konstandinidis, D., and Tsetines, S. (2016). "Pre-earthquake and post-earthquake retrofitting of poorly detailed exterior RC beam-to-column joints." *Engineering Structures*, 109, 1-15.
- Kanakubo, T., and Hosoya, H. (2015). "Bond-Splitting Strength of Reinforced Strain-Hardening Cement Composite Elements with Small Bar Spacing." *ACI Structural Journal*, 112(2), 189.
- Kang, S.-B. (2015). "Structural Behaviour of Precast Concrete Frames Subject to Column Removal Scenarios" PhD Thesis, Nanyang Technological University, Singapore
- Kang, S.-B., Tan, K. H., and Yang, E.-H. (2015). "Progressive collapse resistance of precast beam-column sub-assemblages with engineered cementitious composites." *Engineering Structures*, 98, 186-200.
- Kim, J., and LaFave, J. M. (2009). "Joint shear behavior of reinforced concrete beam-column connections subjected to seismic lateral loading." Newmark Structural Engineering Laboratory. University of Illinois at Urbana-Champaign.
- Kunakubu, T., and Hosoya, H. (2015). "Bond-Splitting Strength of Reinforced Strain-Hardening Cement Composite Elements with Small Bar Spacing." *ACI Structural Journal*, 112(2), 189-198.
- LaFave, J. M., and Kim, J.-H. (2011). "Joint shear behavior prediction for RC beam-column connections." *International Journal of Concrete Structures and Materials*, 5(1), 57-64.
- Lai, L. C. (2012). "Evaluation of high-strength concrete interior beam-column joints under simulated seismic loading." PhD Thesis, Nanyang Technological University, Singapore.
- Li, B., Lam, E. S.-S., Wu, B., and Wang, Y.-y. (2015). "Effect of High Axial Load on Seismic Behavior of Reinforced Concrete Beam-Column Joints with and without Strengthening." *ACI Structural Journal*, 112(6), 713.

- Li, B., Lam, E. S.-S., Wu, B., and Wang, Y.-y. (2015). "Effect of High Axial Load on Seismic Behavior of Reinforced Concrete Beam-Column Joints with and without Strengthening." *ACI Structural Journal*, 112(06), 713-724.
- Li, V. C. (2003). "On Engineered Cementitious Composites (ECC) A Review of the Material and Its Applications " *Journal of Advanced Concrete Technology* 1(3), 215-230.
- Li, V. C., and Fischer, G. (2002). "Reinforced ECC-An Evolution from Materials to Structures." *Proceedings of the 1st fib Congress* Osaka, Japan.
- Li, V. C., and Fischer, G. "Reinforced ECC-An evolution from materials to structures." *Proc., Proceedings of the first FIB congress, Osaka, Japan*, Citeseer, 105-122.
- Li, V. C., Mishra, D. K., Naaman, A. E., Wight, J. K., LaFave, J. M., Wu, H.-C., and Inada, Y. (1994). "On the shear behavior of engineered cementitious composites." *Advanced Cement Based Materials*, 1(3), 142-149.
- Li, V. C., and Wang, S. (2002). "Flexural behaviors of glass fiber-reinforced polymer (GFRP) reinforced engineered cementitious composite beams." *Materials Journal*, 99(1), 11-21.
- Li, V. C., Wang, S., and Wu, C. (2001). "Tensile strain-hardening behavior of polyvinyl alcohol engineered cementitious composite (PVA-ECC)." *Materials Journal*, 98(6), 483-492.
- Lin, C.-M., and Restrepo, J. I. (2002). "Seismic behaviour and design of reinforced concrete interior beam-column joints." *Bulletin of the New Zealand Society for Earthquake Engineering*, 35(2), 108-128.
- Lowes, L. N. (1999). "Finite element modeling of reinforced concrete beam-column bridge connections." University of California, Berkeley.
- Lowes, L. N., and Altoontash, A. (2003). "Modeling Reinforced-Concrete Beam-Column Joints Subjected to Cyclic Loading." *Journal of Structural Engineering*, 129(12), 1686-1697.
- Lowes, L. N., Mitra, N., and Altoontash, A. (2003). *A beam-column joint model for simulating the earthquake response of reinforced concrete frames*, Pacific Earthquake Engineering Research Center, College of Engineering, University of California.

- Lowes, L. N., Mitra, N., and Altoontash, A. (2004). "A Beam-Column Joint Model for Simulating the Earthquake Response of Reinforced Concrete Frames." *PEER Report 2003/10*, Pacific Earthquake Engineering Research Center, University of California, Berkeley.
- Mazzoni, S., McKenna, F., Scott, M. H., and Fenves, G. L. (2006). "OpenSees command language manual." *Pacific Earthquake Engineering Research (PEER) Center*.
- Menegotto, M. "Pinto,(1973), PE, Method of Analysis for Cyclically Loaded Reinforced Concrete Plane Frames Including Changes in Geometry and Non-elastic Behavior of Elements Under Combined Normal Force and Bending." *Proc., IABSE Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well-Defined Repeated Loads, Lisbon*.
- Mitra, N., and Lowes, L. N. (2007). "Evaluation, calibration, and verification of a reinforced concrete beam-column joint model." *Journal of Structural Engineering*, 133(1), 105-120.
- Moehle, J. P., Hooper, J. D. and Lubke, C. D. (2008). "Seismic design of reinforced concrete special moment frames: A guide for practicing engineers." *NEHRP Seismic Design Technical Brief No. 1*.
- Moreno, D. M., Trono, W., Jen, G., Ostertag, C., and Billington, S. L. (2014). "Tension stiffening in reinforced high performance fiber reinforced cement-based composites." *Cement and Concrete Composites*, 50, 36-46.
- Park, R. (2002). "Seismic design and construction of precast concrete buildings in New Zealand." *PCI journal*, 47(5), 60-75.
- Park, R., and Paulay, T. (1975). *Reinforced Concrete Structures*, John Wiley and Sons, New York.
- Park, Y.-J., and Ang, A. H.-S. (1985). "Mechanistic seismic damage model for reinforced concrete." *Journal of structural engineering*, 111(4), 722-739.
- Parra-Montesinos, G., and Wight, J. K. (2000). "Seismic response of exterior RC column-to-steel beam connections." *Journal of structural engineering*, 126(10), 1113-1121.

- Parra-Montesinos, G. J. (2005). "High-performance fiber-reinforced cement composites: an alternative for seismic design of structures." *ACI Structural Journal*, 102(5), 668.
- Parra-Montesinos, G. J., Peterfreund, S. W., and Chao, S.-H. (2005). "Highly damage-tolerant beam-column joints through use of high-performance fiber-reinforced cement composites." *ACI Structural Journal*, 102(3).
- Paulay, T., and Park, R. (1984). *Joints in Reinforced Concrete Frames Designed for Earth-quake Resistance: A Report Prepared for a US-New Zealand-Japan Seminar, 30 July-1 August 1984, Monterey, Calif*, University of Canterbury.
- Pochanart, S., and Harmon, T. (1989). "Bond-Slip Model for Generalized Excitations Including Fatigue." *ACI Material Journal*, 86(5), 465-474.
- Priestley, M., and Paulay, T. (1992). *Seismic design of reinforced concrete and masonry buildings*.
- Priestley, M. N., and MacRae, G. A. (1996). "Seismic tests of precast beam-to-column joint subassemblages with unbonded tendons." *PCI journal*, 41(1), 64-81.
- Qian, S., and Li, V. C. (2007). "Simplified inverse method for determining the tensile strain capacity of strain hardening cementitious composites." *Journal of Advanced Concrete Technology*, 5(2), 235-246.
- Qian, S., and Li, V. C. (2008). "Simplified inverse method for determining the tensile properties of strain hardening cementitious composites (SHCC)." *Journal of Advanced Concrete Technology*, 6(2), 353-363.
- Qudah, S., and Maalej, M. (2014). "Application of Engineered Cementitious Composites (ECC) in interior beam-column connections for enhanced seismic resistance." *Engineering Structures*, 69, 235-245.
- Restrepo, J. I., Park, R., and Buchanan, A. H. (1995). "Tests on connections of earthquake resisting precast reinforced concrete perimeter frames of buildings." *PCI journal*, 40(4), 44-61.
- Said, S. H., and Razak, H. A. (2016). "Structural behavior of RC engineered cementitious composite (ECC) exterior beam-column joints under reversed cyclic loading." *Construction and Building Materials*, 107, 226-234.

- Scott, B., Park, R., and Priestley, M. (1982). "Stress-strain behavior of concrete confined by overlapping hoops at low and high strain rates." *ACI journal*, 79(1), 13-27.
- Shima, H., Chou, L.-L., and Okamura, H. (1987). "Micro and macro models for bond in reinforced concrete." *Journal of the Faculty of Engineering*, 39(2), 133-194.
- Shima, H., Chou, L. L., and Okamura, H. (1987). "Micro and Macro Model for Bond in Reinforced Concrete." *Journal of the Faculty of Engineering, the University of Tokyo (B)*, 39(2), 133-194.
- Shimizu, K., Kanakubo, T., Kanda, T., and Nagai, S. "Shear behavior of steel reinforced PVA-ECC beams." *Proc., 13th World Conference on Earthquake Engineering, Conference Proceedings DVD, Paper*, 8.
- Shin, M., and Lafave, J. M. "Testing and modeling for cyclic joint shear deformations in RC beam-column connections." *Proc., 13th World Conference on Earthquake Engineering*.
- Soltani, M., and Maekawa, K. (2008). "Path-Dependent Mechanical Model for Deformed Reinforcing Bars at RC Interface under Coupled Cyclic Shear and Pullout Tension." *Engineering Structures*, 30(4), 1079-1091.
- Standard, N. Z. (2006). *Concrete Structures Standard: NZS 3101*, NZS.
- Stevens, N., Uzumeri, S., and Collins, M. (1991). "Reinforced concrete subjected to reversed cyclic shear--Experiments and constitutive model." *Structural Journal*, 88(2), 135-146.
- Tsonos, A. G. (2007). "Cyclic load behavior of reinforced concrete beam-column subassemblages of modern structures." *ACI Structural journal*, 104(4), 468.
- Uma, S., and Jain, S. K. (2006). "Seismic design of beam-column joints in RC moment resisting frames-Review of codes." *Structural Engineering and Mechanics*, 23(5), 579.
- van Zijl, G. P. (2007). "Improved mechanical performance: Shear behaviour of strain-hardening cement-based composites (SHCC)." *Cement and Concrete Research*, 37(8), 1241-1247.
- Vecchio, F. J., and Collins, M. P. (1986). "The modified compression-field theory for reinforced concrete elements subjected to shear." *ACI J.*, 83(2), 219-231.

- Viwathanatepa, S., Popov, E. P., and Bertero, V. V. (1979). *Effects of generalized loadings on bond of reinforcing bars embedded in confined concrete blocks*, University of California, Earthquake Engineering Research Center.
- Viwathanatepa, S., Popov, E. P., and Bertero, V. V. (1979). "Effects of Generalized Loadings on Bond of Reinforcing Bars Embedded in Confined Concrete Blocks." *Report No. UCB/EERC-79/22*, College of Engineering, University of California, Berkeley.
- Yang, E.-H., and Li, V. C. (2010). "Strain-hardening fiber cement optimization and component tailoring by means of a micromechanical model." *Construction and Building Materials*, 24(2), 130-139.
- Youssef, M., and Ghobarah, A. (2001). "Modelling of RC beam-column joints and structural walls." *Journal of Earthquake Engineering*, 5(01), 93-111.
- Yu, J. (2012). "Structural Behaviour of Reinforced Concrete Frames Subjected to Progressive Collapse." PhD Thesis, Nanyang Technological University, Singapore.
- Yu, J., and Tan, K. H. (2010). "Macromodel-based simulation of catenary action of RC beam-column sub-assemblages." *Proceedings of the 3rd fib international congress 2010* Washington, DC.
- Yuan, F., Pan, J., and Leung, C. K. Y. (2013). "Flexural behaviors of ECC and concrete/ECC composite beams reinforced with basalt fiber-reinforced polymer." *Journal of Composites for Construction*, 17(5), 591-602.
- Yuan, F., Pan, J., Xu, Z., and Leung, C. K. Y. (2013). "A comparison of engineered cementitious composites versus normal concrete in beam-column joints under reversed cyclic loading." *Materials and Structures*, 46(1-2), 145-159.
- Zhang, R., Matsumoto, K., Hirata, T., Ishizeki, Y., and Niwa, J. (2015). "Application of PP-ECC in beam-column joint connections of rigid-framed railway bridges to reduce transverse reinforcements." *Engineering Structures*, 86, 146-156.

Appendix A: Design of Test Specimens

A.1 Design Information

A six-storey office building with 6 x 4 bay is selected as the prototype structure. The spacing of column in both directions is 5400 mm and the typical storey height is 3600 mm. **Fig. A.1** shows the plan and elevation view of the prototype structure. The size of beams and columns are 300x500 and 500x500 respectively. In the design of the beams and columns, C30/37 concrete is employed. High yield bars (nominal yield strength of 500 MPa) and mild yield bars (nominal strength of 250 MPa) are adopted as longitudinal and transverse reinforcement. The building is designed based on Eurocodes (BS EN 1990:2002, BS EN 1992-1-1:2004 and BS EN 1998-1:2004). For an office building, the estimated of dead and live loads is illustrated as follows.

Live Loads (G_k):

Typical floor = 3.0 kPa

Dead Loads (Q_k):

Imposed dead loads on slab

Concrete Slab (150 mm thick)	=	3.6 kPa
Floor tiles & Finishes	=	0.5 kPa
Frame partitions	=	1.0 kPa
Ceiling	=	0.5 kPa

The basic load combinations are based on BS EN 1990:2002

Combo 1 (gravity): $1.35G_k + 1.50Q_k$

Combo 2 (gravity + seismic): $1.0G_k + 0.3Q_k + 1.0A_{Ed}$

The value of concrete cover is taken as 30mm, thus the effective depth of beam and column is 450 mm. Structural analysis is carried out by ETABS for the two load combinations. Only plane frame D-D is selected for design.

(a) Plan view

(b) Elevation view

Fig. A.1: Plan and elevation view of prototype structure

Seismic Actions

The seismic action is represented by elastic response spectrum, Type 1 ($M_s > 5.5$) for soil B, the reference peak ground acceleration amounts to $a_{gR} = 0.25g$. As indicated in BS EN 1998-1:2004 (Figure 3.1 and Table 3.2), $S = 1.2$, $T_B = 0.15s$, $T_C = 0.5s$, $T_D = 2.0s$. The building is classified as importance class II, therefore the peak ground acceleration $a_g = 1.0 \times a_{gR} = 0.25g$. Based on BS EN 1998-1:2004 (Section 3.2.2.2), the value of elastic response spectrum $S_e(T)$ is defined with taking the value of $\eta=1$ for 5% viscous damping. **Fig. A.2** illustrates the elastic response spectrum adopted in ETABS.

Fig. A.2: Elastic response spectrum (Type 1)

A.2 Gravity Design

A.2.1 Design of Beam

The continuous beams at level 1 from grid line 2 to grid line 4 is selected for design, the design value of bending moment and shear force are shown in **Table A.1**.

Table A.1: Design value of bending moment and shear force in gravity design

	Support 2	Midspan	Support 3	Midspan	Support 4
Design Moment, M_U (kNm)	-139.60	98.74	-139.60	98.74	-139.60
Design Shear Force, V_U (kN)	143.02	-	± 142.33	-	-143.02

From BS EN 1992-1-1:2004 (Section 9.2.1.1), the minimum amount of tensile reinforcement provided for beam is

$$A_{s,min} = 0.26f_{ctm}bb_w d/f_{yk} = 0.26 \times 2.9 \times 300 \times 450/500 = 204 \text{ mm}^2$$

$$\text{but not less than } 0.0013b_w d = 175.5 \text{ mm}^2$$

$$A_{s,max} = 0.04A_c = 6000 \text{ mm}^2$$

Table A.2 shows the design of flexural reinforcement for beams. The provided amount of tensile reinforcements are satisfying the maximum and minimum area as specified.

Table A.2: Design calculation of flexural reinforcement

	Equation	Support 2	Midspan	Support 3	Midspan	Support 4
Mu (kNm)	-	139.60	98.74	139.60	98.74	139.60
K	$M/(bd^2 f_{ck})$	0.077	0.054	0.077	0.054	0.077
Z (mm)	$d \left[0.5 + \sqrt{(0.25 - K/1.134)} \right]$	417.03	427.44	417.03	427.44	417.03
$A_{s,req'd}$ (mm ²)	$M/(0.87f_{yk}z)$	770	531	770	531	770
Bar provided	-	3H20	2H20	3H20	2H20	3H20
$A_{s,pro'd}$ (mm ²)	$n \times \pi/4 \times d \times d$	943	628	943	628	943

Design moment	$0.87f_{yk}A_s(d - s/2)$	171	117	171	117	171
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The maximum design shear is taken as 143.33 kN as indicated in **Table A.1**. From BS EN 1992-1-1:2004 (Section 6.2.2), the shear strength provided by concrete can be determined by:

$$V_{Rd,c} = [C_{Rd,c}k(100\rho_1f_{ck})^{1/3} + k_1\sigma_{cp}]b_wd$$

$$\text{Where } C_{Rd,c} = 0.18/\gamma_c = 0.18/1.5 = 0.12$$

$$k = 1 + \sqrt{200/d} = 1.667$$

$$\rho_1 = A_{s1}/(b_w)d = 943/300 \times 450 = 0.007$$

$$k_1 = 0.15, \sigma_{cp} = 0$$

$$V_{Rd,c} = [0.12 \times 1.667 \times (100 \times 0.007 \times 30)^{1/3} + 0] \times 300 \times 450 \times 10^{-3} = 74.51 \text{ kN}$$

It is less than the maximum shear force 143.33 kN, thus shear reinforcement is required. According to BS EN 1992-1-1:2004 Section 6.2.3, for members with vertical shear reinforcement, the shear resistant is

$$V_{Rd,s} = A_{sw}zf_{ywd} \cot\theta/s \text{ in which } z = 0.9d \text{ and}$$

The recommended limiting values for $\cot\theta$ is $1 \leq \cot\theta \leq 2.5$

Assuming $\cot\theta = 2.5$,

$$\frac{A_{sw}}{s} = \frac{V_{Rd,s}}{zf_{ywd}\cot\theta} = \frac{143.33 \times 10^3}{0.9 \times 450 \times 250/1.15 \times 2.5} = 0.65 \text{ mm}^2/\text{mm}$$

Try R12, two-leg stirrups, $A_{sw} = 226 \text{ mm}^2$, $s = 200 \text{ mm}$

$$\frac{A_{sw}}{s} = \frac{226}{200} = 1.13 \text{ mm}^2/\text{mm} > 0.65 \text{ (ok)}$$

According to BS EN 1992-1-1:2004 (Section 9.2.2), the ratio of shear reinforcement is supposed to be larger than:

$$\rho_{w,min} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} = 0.08 \times \sqrt{30}/250 = 0.00175$$

For the proposed shear reinforcement,

$$\rho_w = \frac{A_{sw}}{(sb_w)} = \frac{226}{200 \times 300} = 0.00377 > 0.00175(OK)$$

The maximum longitudinal spacing between shear reinforcement should not exceed:

$$S_{l,max} = 0.75d(1 + \cot \alpha) = 0.75 \times 450 = 337.5 \text{ mm}$$

The provided stirrup spacing is smaller than 337.5 mm, OK.

Thus, transverse reinforcement R12-200 mm c/c is adopted for beams.

A.2.2 Design of Column

The internal column is design to axial compression, shear force and bending moment can be neglected due to their negligibly values. Column at the middle of plane frame D-D is selected to represent the interior joint specimen. **Table A.3** shows the design of column.

Table A.3: Design of column for axial compression

Ultimate axial load, N (kN)	4002.60
Bars (for axial)	8H20
As provided (mm ²)	2514

Axial capacity of column,

$$N = 0.567f_{ck} \cdot A_c + 0.87f_{yk} \cdot A_{sc}$$

Net concrete area, $A_c = 500^2 - 2514 = 247,486 \text{ mm}^2$

Therefore,

$$N = (0.567 \times 30 \times 247,486) + (0.87 \times 500 \times 2514)/10^3 = 5303.3 \text{ kN} >$$

$N (ok)$

From BS EN 1992-1-1:2004 (Section 9.5.2), the total amount of longitudinal reinforcement in column should not be less than:

$$A_{s,min} = \frac{0.10N_{Ed}}{f_{yd}} = \frac{0.10 \times 4002.60 \times 1000 \times 1.15}{500} = 921\text{mm}^2$$

Or $0.002A_c = 0.002 \times 500 \times 500 = 500\text{mm}^2$ whichever is greater

The area of longitudinal reinforcement should not exceed:

$$A_{s,max} = 0.04A_c = 0.04 \times 500 \times 500 = 10,000 \text{ mm}^2$$

Hence the longitudinal reinforcing bars provided satisfy the maximum and minimum requirements in design code.

Under gravity loading condition, minimum shear reinforcement is to be provided in column.

$$\rho_{w,min} = 0.08\sqrt{f_{ck}}/f_{yk} = 0.08 \times \sqrt{30}/250 = 0.00175$$

Try R12, two-leg stirrups, $A_{sw} = 226 \text{ mm}^2$, $s = 200 \text{ mm}$

$$\rho_w = \frac{A_{sw}}{sb_w} = \frac{226}{200 \times 500} = 0.00226 > 0.00175 \text{ (ok)}$$

Diameter of transverse reinforcement should be at least one-quarter of longitudinal bar or 6mm whichever is greater. The spacing of transverse reinforcement should not exceed either 400 mm or the least dimension of the column cross section 500 mm.

Thus, the proposed stirrup diameter and spacing meet the detailing requirement.

Therefore, 8H20 longitudinal bars and R12@200 two-leg stirrups are adopted in column design.

A.3 Seismic Design

This office building is design to medium ductility class of structures (DCM). The geometrical constraints and materials have been checked to meet the requirements as specified in BS EN 1998-1:2004, section 5.4.1.1 and 5.4.1.2. **Table A.4** shows the checking on geometrical and materials requirements.

Table A.4: Geometrical constraints and material requirements in DCM

Geometrical constraints	Check
Beam: $b_w < \min \{b_c + h_w; 2b_c\}$, $b_c=500$, $h_w=500$	$b_w=300 < 1000$ (ok)
Column: Unless $\theta < 0.1$, the cross-section $>$ one tenth of the larger distance between the point of contra-flexure and end of column	$500 > 1/10 * 2700$ $=270$ (ok)
Material requirements	Check
Concrete $<$ C16/20 shall not be used in primary seismic elements	C30/37 (ok)
Only ribbed bars shall be used as reinforcing bar in critical regions, in exception to close stirrups	(ok)

A.3.1 Design of Beam

Similarly, continuous beams at level 1 from grid line 2 to grid line 4 is selected for design, the design value of bending moment and shear force are shown in **Table A.5**.

Table A.5: Design value of bending moment and shear force in seismic design

	Support 2	Midspan	Support 3	Midspan	Support 4
Design Moment, M_U (kNm)	-158.76	63.39	-167.30	63.39	-158.76
Design Shear Force, V_U (kN)	120.31	-	± 120.31	-	-120.31

Flexural design

From BS EN 1992-1-1:2004 (Section 9.2.1.1), the minimum amount of tensile reinforcement provided for beam is:

$$A_{s,min} = 0.26f_{ctm}bb_w d/f_{yk} = 0.26 \times 2.9 \times 300 \times 450/500 = 204 \text{ mm}^2$$

$$\text{but not less than } 0.0013b_w d = 175.5 \text{ mm}^2$$

$$A_{s,max} = 0.04A_c = 6000 \text{ mm}^2$$

Table A.6: Design calculation of flexural reinforcement in seismic design

	Equation	Support 2	Midspan	Support 3	Midspan	Support 4
Mu (kNm)	-	158.76	63.39	167.30	63.39	158.76
K	$M/(bd^2f_{ck})$	0.087	0.034	0.092	0.034	0.087
Z (mm)	$d \left[0.5 + \sqrt{(0.25 - K/1.134)} \right]$	412.32	435.75	409.5	435.75	412.32
$A_{s,req'd}$ (mm ²)	$M/(0.87f_{yk}z)$	885	334	939	334	885
Bar provided	-	3H25	3H25	3H25	3H25	3H25
$A_{s,pro'd}$ (mm ²)	$n \times \pi/4 \times d \times d$	1472	1472	1472	1472	1472
Design moment	$0.87f_{yk}A_s(d - s/2)$	263	263	263	263	263

The provided amounts of tensile reinforcements are satisfying the maximum and minimum area as specified.

Shear design

Derive shear demand from flexural capacity, hogging capacity can be calculated as

$$M_{Rd,Top} = 63.39 \times 1472/334 = 279 \text{ kNm}$$

And sagging capacity is

$$M_{Rd,Bottom} = 167.30 \times 1472/939 = 263 \text{ kNm}$$

Shear force,

$$V_{E,d} = \gamma_{Rd} [M_{Rd,Top} + M_{Rd,Bottom}]/l_{cl} + V_g$$

For DCM structures, $\gamma_{Rd} = 1.0$ in beams and from ETABS, $V_g = 120.31 \text{ kN}$

$$l_{cl} = 5.4 - 0.5 = 4.9 \text{ m}$$

Thus

$$V_{E,d} = 1.0 [279 + 263]/4.9 + 120.31 = 230.92 \text{ kN}$$

Shear strength provided by concrete, $V_{Rd,c} = 74.51 \text{ kN}$ (as in gravity design).

Therefore shear reinforcement is required.

Assuming $\cot\theta = 2.5$

$$\frac{A_{sw}}{S} = \frac{V_{Rd,s}}{z f_{ywd} \cot\theta} = \frac{230.92 \times 10^3}{0.9 \times 450 \times \frac{250}{1.15} \times 2.5} = 1.05 \text{ mm}^2/\text{mm}$$

Try R12, two-leg stirrups, $A_{sw} = 226 \text{ mm}^2$, $s = 200 \text{ mm}$

$$\frac{A_{sw}}{S} = \frac{226}{200} = 1.13 \text{ mm}^2/\text{mm} > 1.05 \text{ (ok)}$$

Hence, R12-200 stirrups are adopted for shear reinforcement in the beam.

Detailing for local ductility

All the requirements of detailing for local ductility are based on BS EN 1998-1:2004, Section 5.4.3.1.2.

Flexural bars:

The regions of a primary seismic beam up to distance $l_{cr}=h_w$ shall be considered as critical region.

$$l_{cr} = 500 \text{ mm}$$

From BS EN 1998-1:2004, equation 5.11, the reinforcement ratio at the tension zone does not exceed the value of:

$$\rho_{max} = \rho' + \frac{0.0018 f_{cd}}{\mu_\phi \epsilon_{sy,d} f_{yd}}$$

where: $\mu_\phi = 2q_0 - 1 = 2(3.9) - 1 = 6.8$ and $q_0 = 3 \times 1.3$ (for frame) = 3.9

$$\epsilon_{sy,d} = \frac{f_{yk}/\gamma_s}{E_s} = \frac{500/1.15}{200 \times 10^3} = 0.0022$$

Area of reinforcement in compression zone = 628 mm^2 , thus

$$\rho' = \frac{1472}{300 \times 450} = 0.00868$$

Therefore

$$\rho_{max} = 0.00868 + \frac{0.0018}{6.8 \times 0.0022} \frac{20}{435} = 1.42\%$$

By inspection, the proposed reinforcement ration at the tension face,

$$\rho = 1472/300 \times 450 = 1.09\% < \rho_{max} \text{ (ok)}$$

From BS EN 1998-1:2004, equation 5.11, the reinforcement ratio of tension zone along the entire length of primary seismic beam shall not be less than the value of:

$$\rho_{min} = 0.5 \frac{f_{ctm}}{f_{yk}} = 0.5 \frac{2.9}{500} = 0.29\% < 1.09\% \text{ (ok)}$$

The proposed flexural bars are satisfying all the requirements of local ductility.

Transverse bars:

Within the critical regions of primary seismic beams, hoops provided must satisfy the following conditions:

- i) The diameter d_{bw} of the hoops shall not be less than 6mm. (R12 is adopted, ok).
- ii) According to BS EN 1998-1:2004, equation 5.13, the spacing shall not exceed:

$$s = \min\{h_w/4; 24d_{bw}; 225; 8d_{bL}\} = \min\{\frac{500}{4}; 24 \times 12; 225; 8 \times 20\} \\ = 125 \text{ mm}$$

(Spacing of 100mm is used in critical region, ok)

- iii) The first hoop shall be placed not more than 50 mm from the beam end section.

Outside the critical region, transverse bars shall satisfy the requirements as specified in BS EN 1992-1-1:2004 Section 9.2.2:

- i) The ratio of shear reinforcement is supposed to be larger than:

$$\rho_{w,min} = 0.08 \sqrt{f_{ck}} / f_{yk} = 0.08 \times \sqrt{30} / 250 = 0.00175$$

For the proposed shear reinforcement,

$$\rho_w = \frac{A_{sw}}{(sb_w)} = \frac{226}{200 \times 300} = 0.00377 > 0.00175(OK)$$

- ii) The maximum longitudinal spacing between shear reinforcement should not exceed:

$$S_{l,max} = 0.75d(1 + \cot \alpha) = 0.75 \times 450 = 337.5 \text{ mm}$$

The provided stirrup spacing is 200mm < 337.5 mm (ok)

Anchorage of reinforcement in beam:

According to BS EN 1998-1:2004, section 5.6.2.2,

- i) The part of beam longitudinal reinforcement bent in joints for anchorage shall always be placed inside the corresponding column hoops.
- ii) To prevent bond failure the diameter of beam longitudinal bars passing through beam-column joints, d_{bL} , shall be limited in accordance with the equation 5.50a:

$$\frac{d_{bL}}{h_c} \leq \frac{7.5f_{ctm}}{\gamma_{Rd}f_{yd}} \cdot \frac{1 + 0.8v_d}{1 + 0.75k_D \cdot \rho' / \rho_{max}}$$

where

$$v_d = \frac{N_{ed}}{f_{cd} \cdot A_c} = \frac{2438 \times 10^3}{20 \times 500^2} = 0.488$$

For DCM,

$$\gamma_{Rd} = 1.0 \text{ and } k_D = 2/3$$

Therefore

$$\frac{d_{bL}}{500} \leq \frac{7.5 \times 2.9}{1.0 \times 435} \cdot \frac{1 + 0.8(0.488)}{1 + 0.75(2/3)(0.00868/0.0142)}$$

$$\frac{d_{bL}}{500} \leq \frac{21.75}{435} \cdot \frac{1.39}{1.31}$$

$$d_{bL} \leq 26.53 \text{ mm}$$

The proposed bar diameter is 25mm < 26.53 mm, Ok.

- iii) The top or bottom bars passing through interior joints, shall terminate in the members framing into the joint at a distance not less than l_{cr} from the face of the joint.

Therefore R12-100 hoops (two-legs) are used within the critical regions of beam whereas R12-200 hoops (two-legs) are used outside the critical regions of beam.

A.3.2 Design of Column

The frame is a moment frame in both directions and it is designed for uniaxial bending about each direction rather than biaxial bending. However, the uniaxial capacity is reduced by 30 percent as specified in BS EN 1998-1:2004, section 5.4.3.2.1.

Design of longitudinal bars

The sum of the design values of the moment resistance of the beams framing into the joint,

$$\sum M_{Rb} = 263 + 263 = 526 \text{ kNm}$$

Assume 45 percent and 55 percent moment distribution to the upper and lower column section, design value of the column moment resistance,

$$M_{Rc1} = 0.55 \times 526 = 289 \text{ kNm}$$

Axial load from analysis,

$$N = 2438 \text{ kN}$$

Check normalised axial compression,

$$v_d = \frac{N_{ed}}{f_{cd} \cdot A_c} = \frac{2438 \times 10^3}{20 \times 500^2} = 0.488 < 0.65 \text{ (for DCM)(ok)}$$

Check column resistances by using design chart,

$$\frac{N_{Ed}}{bh f_{ck}} = \frac{2438 \times 10^3}{500 \times 500 \times 30} = 0.325$$

$$\frac{M'_y}{bh^2 f_{ck}} = \frac{176 \times 10^6 / 0.7}{500 \times 500^2 \times 30} = 0.11$$

$$\frac{d}{h} = \frac{450}{500} = 0.9$$

From design chart (**Fig. A.3**), $A_s f_{yk} / b h f_{ck} = 0$, select 0.15

$$A_s = 0.15 \times 500 \times 500 \times 30 / 500 = 2250 \text{ mm}^2$$

Provide 8H25 longitudinal bars, $A_s = 3927 \text{ mm}^2$

Check capacity for maximum compression,

$$\frac{A_s f_{yk}}{b h f_{ck}} = \frac{3927 \times 500}{500 \times 500 \times 30} = 0.2618$$

$$\frac{N_{Ed}}{b h f_{ck}} = \frac{2438 \times 10^3}{500 \times 500 \times 30} = 0.325$$

From column design chart (**Fig. A.3**),

$$\frac{M'_y}{b h^2 f_{ck}} = 0.15$$

$$M_{cap} = 0.15 \times 500 \times 500^2 \times 30 \times 10^{-6} = 562.5 \text{ kNm}$$

Fig. A.3: Column design chart

Design of transverse bars

For a conservative design, the column shear could be based upon the flexural capacity at maximum compression calculated above. However, the column flexural capacities is allowed to be multiplied by the ratio of $\sum M_{Rb}/\sum M_{Rc}$.

Shear force,

$$V_{E,d} = \gamma_{Rd} \times (\sum M_{Rb}/\sum M_{Rc}) \times (M_{c,Top} + M_{c,Bottom})/l_{cl}$$

For DCM structures, $\gamma_{Rd} = 1.1$ and $l_{cl} = 3.6 - 0.5 = 3.1$ m

Thus

$$V_{E,d} = 1.1 [526/(2 \times 562.5)] \times (562.5 + 562.5)/3.1 = 186.66 \text{ kN}$$

Assuming $\cot\theta = 2.5$,

$$\frac{A_{sw}}{S} = \frac{V_{Rd,s}}{z f_{ywd} \cot\theta} = \frac{186.66 \times 10^3}{0.9 \times 450 \times (250/1.15) \times 2.5} = 0.85 \text{ mm}^2/\text{mm}$$

Try R12, two-leg stirrups, $A_{sw} = 226 \text{ mm}^2$, $s = 200 \text{ mm}$

$$\frac{A_{sw}}{S} = \frac{226}{200} = 1.13 \text{ mm}^2/\text{mm} > 0.85 \text{ mm}^2/\text{mm} \text{ (ok)}$$

Provide R12 at spacing 200mm as transverse bars.

Detailing for local ductility

All the requirements of detailing for local ductility are based on BS EN 1998-1:2004, Section 5.4.3.2.2,

Critical region for column is defined as equations 5.14,

$$l_{cr} = \max\{h_c; l_{cl}/6; 0.45\}$$

$$l_{cr} = \max\{500; 3100/6; 0.45\}$$

$$l_{cr} = 516.7 \text{ mm}$$

If $l_c/h_c < 3$, entire height shall be considered as critical region.

$$l_c/h_c = 3600/500 = 7.2 > 3 \text{ (ok)}$$

Longitudinal bars:

The total longitudinal reinforcement ratio should be between the value of:

$$0.01 < \rho_1 < 0.04$$

Provided Provide 8T25 longitudinal bars, $\rho_1 = 3927/500 \times 450 = 0.017$ (ok)

According to BS EN 1998-1:2004, Section 5.4.3.2.2(2), at least one intermediate bar shall be provided between corner bars along each column side to ensure the integrity of the beam-column joints. Thus 3 bars of T25 are placed at each column side to form a total of 8T25.

Transverse bars:

Within the critical regions of primary seismic columns, hoops provided must satisfy the following conditions:

- i) The diameter of hoops shall be at least 6 mm. (R12 is adopted, ok). The hoop pattern shall be such that the cross section benefit from the triaxial stress.

- ii) The spacing, s , of hoops does not exceed:

$$s = \min \left\{ \frac{b_o}{2}; 175; 8d_{bL} \right\} = \min \left\{ \frac{450}{2}; 175; 8 * 25 \right\} = 175$$

- iii) The distance between consecutive longitudinal bars engaged by hoops does not exceed 200mm.

$$\begin{aligned} \text{Distance between restrained main bars} &= \frac{\{500 - (2 \times 50) - 25\}}{2} \\ &= 187.5 \text{ mm (ok)} \end{aligned}$$

Outside the critical regions of primary seismic columns, hoops provided must satisfy the following conditions:

- i) The diameter of hoops shall be at least 6 mm or one-quarter of longitudinal bar whichever is greater.

$$d_{bw} = \max \left\{ \frac{25}{4}; 6 \right\} = 6.25 \text{ mm}$$

- ii) The spacing of hoops should not exceed:

$$s = \min \{h_c; b_c; 400; 20d_{bL}\} = \min \{500; 500; 400; 500\} = 400$$

Confinement reinforcement in beam-column joint:

For DCM, confinement reinforcement within beam-column joint and in the critical regions at the base of a column must meet the provisions of BS EN 1998-1:2004, Section 5.4.3.2.2(8)-(11). In addition to the Section 5.4.3.2.2(10)-(11) as mentioned above (A.3.2.3.2), additional requirements of clauses 5.4.3.2.2 (8) and (9) need to be checked. According to Equation 5.15,

$$\alpha\omega_{wd} \geq 30\mu_{\phi}v_d\epsilon_{sy,d}\frac{b_c}{b_o} - 0.035$$

Where α is the confinement effectiveness factor,

$$\alpha = \alpha_n \cdot \alpha_s \text{ where } \alpha_n = 1 - \sum (b_i)^2 / 6b_o h_o \text{ and } \alpha_s = (1 - s/2b_o)(1 - s/2h_o)$$

$$b_o = h_o = 428 \text{ mm}$$

For all main column bars equally spaced:

$$b_i = (b_o - 25)/2 = 201.5 \text{ mm}$$

$$\alpha_n = 1 - 8(201.5)^2 / 6 \times 428 \times 428 = 0.7$$

$$\alpha_s = (1 - 100/856)^2 = 0.78$$

$$\alpha = 0.78 \times 0.7 = 0.55$$

And ω_{wd} is the mechanical volumetric ratio of confining hoops within the critical region:

$$\omega_{wd} = [(A_{svx}/b_o s) + (A_{svy}/bh_o s)] \cdot (f_{yd}/f_{cd})$$

$$\omega_{wd} = [2(226/428 \times 100)] \cdot (435/20) = 0.23$$

As before

$$\mu_{\phi} = 6.8, \epsilon_{sy,d} = 0.0022 \text{ and } v_d = 0.488$$

$$b_c/b_o = 500/428 = 1.17$$

$$30\mu_{\phi}v_d\epsilon_{sy,d}\frac{b_c}{b_o} - 0.035 = (30 \times 6.8 \times 0.488 \times 0.0022 \times 1.17) - 0.035 = 0.22$$

Therefore

$$\alpha\omega_{wd} = 0.55 \times 0.23 = 0.13 < 0.22 \text{ (not sufficient)}$$

Consider 16 mm diameter hoops at spacing 100mm, $A_s = 402 \text{ mm}^2$

$$\omega_{wd} = [2(402/428 \times 100)]. (435/20) = 0.41$$

$$\alpha\omega_{wd} = 0.55 \times 0.41 = 0.23 > 0.22 \text{ (ok)}$$

From clauses 5.4.3.2.2 (9), check the minimum value of ω_{wd} :

$$\omega_{wd} = \frac{0.22}{0.55} = 0.4 > 0.08 \text{ (ok)}$$

Therefore R16-100 hoops (two-legs) are used within the critical regions of column (including beam-column joint) whereas R12-200 hoops (two-legs) are used outside the critical regions of column.

A.4 Mapping of Prototype to Model

To facilitate the experimental test, the prototype frame is scaled down to one-half. However, the reinforcement ratio was kept the same as the prototype building. Thus, the dimensions of beams and columns were 150 mm by 250 mm and 250mm by 250 mm, respectively. **Table A.7** and **Table A.8** show the reinforcement details of prototype and scaled-down model of beams and columns, respectively for gravity design. **Table A.9** and **Table A.10** show reinforcement details of prototype and scaled-down model of beams and columns, respectively for seismic design.

Table A.7: Comparison of reinforcement details between prototype and model beam (gravity design)

	Top reinforcement	Bottom reinforcement	Mid-span stirrups
Prototype beam	3H20 (943 mm ²)	2H20 (628 mm ²)	R12@200 (226 mm ²)
	$\rho = \frac{A_s}{b_w d}$ $= \frac{943}{300 \times 450}$ $= 0.699\%$	$\rho = \frac{A_s}{b_w d}$ $= \frac{628}{300 \times 450}$ $= 0.465\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{226}{300 \times 200}$ $= 0.377\%$
Model beam	3H10 (236 mm ²)	2H10 (157 mm ²)	R6@100 (56.6 mm ²)
	$\rho = \frac{A_s}{b_w d}$ $= \frac{236}{150 \times 220}$ $= 0.715\%$	$\rho = \frac{A_s}{b_w d}$ $= \frac{157}{150 \times 220}$ $= 0.476\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{56.6}{150 \times 100}$ $= 0.377\%$

Table A.8: Comparison of reinforcement details between prototype and model column (gravity design)

	Longitudinal reinforcement	Stirrups
Prototype column	8H20 (2514 mm ²)	R12@200 (226 mm ²)
	$\rho = \frac{A_s}{b_w d}$ $= \frac{2514}{500 \times 450}$ $= 1.117\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{226}{500 \times 200}$ $= 0.226\%$
Model column	8H10 (628 mm ²)	R6@100 (56.6 mm ²)
	$\rho = \frac{A_s}{b_w d}$ $= \frac{628}{250 \times 220}$ $= 1.142\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{56.6}{250 \times 100}$ $= 0.226\%$

Table A.9: Comparison of reinforcement details between prototype and model beam (seismic design)

	Top reinforcement	Bottom reinforcement	Mid-span stirrups	Beam end Stirrups (critical region)
Prototype beam	3H25 (1472 mm ²)	3H25 (1472 mm ²)	R12@200 (226 mm ²)	R12@100 (226 mm ²)
	$\rho = \frac{A_s}{b_w d}$ $= \frac{1472}{300 \times 450}$ $= 1.090\%$	$\rho = \frac{A_s}{b_w d}$ $= \frac{1472}{300 \times 450}$ $= 1.090\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{226}{300 \times 200}$ $= 0.377\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{226}{300 \times 100}$ $= 0.753\%$
Model beam	3H13 (398 mm ²)	3H13 (398 mm ²)	R6@100 (56.6 mm ²)	R6@50 (56.6 mm ²)
	$\rho = \frac{A_s}{b_w d}$ $= \frac{398}{150 \times 220}$ $= 1.206\%$	$\rho = \frac{A_s}{b_w d}$ $= \frac{398}{150 \times 220}$ $= 1.206\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{56.6}{150 \times 100}$ $= 0.377\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{56.6}{150 \times 50}$ $= 0.755\%$

Table A.10: Comparison of reinforcement details between prototype and model column (Seismic design)

	Longitudinal reinforcement	Stirrups	Stirrups (critical region)
Prototype column	8H25 (3926 mm ²)	R12@200 (226 mm ²)	R16@100 (402 mm ²)
	$\rho = \frac{A_s}{b_w d}$ $= \frac{3926}{500 \times 450}$ $= 1.744\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{226}{500 \times 200}$ $= 0.226\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{402}{500 \times 100}$ $= 0.804\%$
Model column	8H13 (1061 mm ²)	R6@100 (56.6 mm ²)	R8@50 (100.5 mm ²)
	$\rho = \frac{A_s}{b_w d}$ $= \frac{628}{250 \times 220}$ $= 1.929\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{56.6}{250 \times 100}$ $= 0.226\%$	$\rho = \frac{A_s}{b_w s}$ $= \frac{100.5}{250 \times 50}$ $= 0.804\%$