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**Experimental and Analytical Investigations on
Seismic Behavior of Reinforced Concrete Frames
and Members using High-Strength Materials**

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Experimental and Analytical Investigations on Seismic Behavior of Reinforced Concrete Frames and Members using High-Strength Materials

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ABSTRACT

The increasing use of high-strength concrete (HSC) in the construction industry is due to the numerous benefits offered by HSC compared to the ordinary-strength concrete. The use of HSC is prevalent in the columns of tall reinforced-concrete (RC) structures in cities located at high-seismic zones. The privilege of utilizing HSC in building structures is more significant in those structural members that are dominated by axial loading behavior. For a given load, the columns constructed using HSC require smaller cross-sectional area, which will increase the available space in each building story. The coupling of HSC with high-strength steel (HSS) reinforcement also appears to be an attractive proposition in the design of RC members. The congestion issue of reinforcing bars and hoops in a beam-column joint region can be improved by using HSS bars. However, there are several issues and concerns regarding the extensive use of high-strength materials. Investigation will be done to verify if HSC members and frames reinforced with HSS can be designed to have a predictable strength and to behave in a ductile manner under severe seismic loading. Most of the current building codes and standards do not address design provisions for concrete with a compressive strength of greater than 55 MPa and the reinforcing steel with the yield strength of larger than $f_y = 550$ MPa. The aim of this research program is to investigate the inelastic behavior of HSC columns and beam-column joints reinforced with high-strength steel.

Experimental and analytical investigations were carried out on HSC beam-column joints with Grade 700 reinforcing longitudinal bars in beams and columns. Seven interior and five exterior full-scale RC beam-column joints with various reinforcement detailing were designed according to the special seismic provisions of ACI 318-11 code. The variables in test specimens include the concrete compressive strength, the reinforcement yield strength, and the level of axial compression loading. The test specimens were subjected to constant column axial loading as well as the quasi-static lateral loading reversals. The performance of each test assembly was examined in terms of cracking patterns, lateral loading capacity,

strain profiles of the reinforcement, secant stiffness, energy dissipation capacity, joint shear strength, and the bond performance. All specimens displayed ductile failure mode and it was concluded that the use of HSC disregarding the presence of axial compression loading could improve the bond condition of the specimens. Parametric studies via finite-element analysis (FEA) were performed to study the influence of various parameters on the strength, bond condition, and energy dissipation capacity of the specimens. In general, the effect of parameters such as material properties, the axial compression loading level, and the effect of the development length of beam bars passing through the joints were investigated. Based on the experimental findings and the analytical parametric studies, several design recommendations were proposed. In addition, a backbone curve model for the shear behavior of joints was proposed. Furthermore, the bond stress distribution along the beam bars in joints was proposed and calibrated based on the experimental result. This bond distribution was incorporated in an existing strut-and-tie model to study the force flow mechanism in beam-column joints. A comparison was made between the observed cracking pattern of test specimens and the predicted cracking pattern based on strut-and-tie modeling.

Furthermore, the effectiveness of a column design method that uses longitudinal reinforcement with various steel grades was investigated. Based on analytical investigations, the influence of various design parameters on the ultimate drift capacity of RC columns was studied and new drift-based design provisions and charts were proposed. These models can be used as a design tool for HSC columns reinforced with mixed-grade longitudinal steel bars.

LIST OF PUBLICATIONS

1. **Alaee P**; Li B, 2016, 'High-Strength Concrete Interior Beam-Column Joints with High-Yield-Strength Steel Reinforcements', Journal of Structural Engineering, DOI: 10.1061/(ASCE)ST.1943-541X.0001773
2. **Alaee, P.**, Li, B. and Cheung, P.P.C. (2015), “Parametric investigation of 3D RC beam–column joint mechanics”, Magazine of Concrete Research, Vol. 67, No. 19, pp. 1054 - 1069, DOI: 10.1680/macr.15.00005.
3. **Alaee, P.** and Li, B. (2015), “Seismic Behavior of HSC Beam-Column Joints with High-Yield Strength Steel Reinforcements”, Proceedings of 10th Pacific Conference on Earthquake Engineering, Sydney, Australia, 6-8 November 2015.

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LIST OF NOTATIONS

a_c	Depth of column compression zone
a_i	The i^{th} factor in Equation (7.1)
a_v	Shear span
A_c	Cross-sectional area of the RC column measured out-to-out of the transverse reinforcement
A_g	Gross cross-sectional area of the RC column
A_j	Effective area of the joint
A_{jh}	Total area of effective horizontal joint shear reinforcement
A_s	Area of tension reinforcement
A'_s	Area of compression reinforcement
A_{sh}	Total section area of rectangular hoops in the joint
A_{sc}	Total area of longitudinal reinforcement of column
A_{sv}	Area of shear reinforcement
b	Width of beam
b_c	Width of column
b_j	Effective width of joint
BI	Beam reinforcement index
c	Cohesion factor

$C_{b,c}$	Concrete compressive force of beam
$C_{b,s}$	Compressive force of beam bar
C_j	Factor in Equation (2.41)
d	Distance from extreme compression fiber to the centroid of tension reinforcement
d_b	Nominal diameter of the reinforcing bar
d_b	Effective depth of the beam in Equation (2.3)
D	Compression force in the diagonal strut
$D_{p=0}$	Ultimate drift capacity of RC columns with zero axial loading
D_{ult}	Ultimate drift capacity of RC columns
D.R.	Story drift ratio
E_1	Peak lateral resistance for positive loading for third cycle of loading sequence
E_2	Peak lateral resistance for negative loading for third cycle of loading sequence
E_c	Modulus of elasticity of concrete
E_s	Modulus of elasticity of steel reinforcement
f_c	Diagonal compressive stress
$f_{c,s}$	Average uniaxial stress in the center of the joint panel
f_{cx}	Average concrete stress in the x direction
f_{cy}	Average concrete stress in the y direction
f'_c	Concrete compressive strength

f_t	Tension strength of concrete
f_{ds}	Yield stress of steel reinforcement in Von-Mises yield criterion
$^t f_{ext}$	Factor in Newton-Raphson method
$^{t+\Delta t} f_{ext}$	Factor in Newton-Raphson method
$f_{int,1}$	Factor in Newton-Raphson method
f_u	Ultimate strength of steel reinforcement
f_y	Yield strength of steel reinforcement
f_{yb}	Yield strength of the steel reinforcement in beam
f_{yc}	Yield strength of the steel reinforcement in column
f_{yh}	Yield strength of the transverse reinforcement
f_{yHs}	Yield strength of the high-strength steel reinforcement
f_{yv}	Yield strength of shear reinforcement
F_h	Tension force in the horizontal tie
F_v	Tension force in the vertical tie
g_1	Factor in Newton-Raphson method
G	Pre-cracking initial shear stiffness in total strain crack models
G_f	Fracture energy of concrete
G^{cr}	Post-cracking shear stiffness in total strain crack models
h	Height of the beam specimen in RILEM test
h	Story height or vertical distance between the column end pins

h_b	Beam depth
h_c	Column depth
h_i	Distance between top and bottom linear potentiometers in the region i of beam
h_j	Effective depth of joint
h_n	Notch height in RILEM test
j	Lever arm of the moment in beam
K	Initial stiffness for positive loading for first cycle
K'	Initial stiffness for negative loading for first cycle
k_2	Confinement efficiency parameter
l_{bd}	Design anchorage length in Eurocode 2
$l_{b,rqd}$	Basic required anchorage length in Eurocode 2
l_{dh}	Development length of longitudinal beam bars
L	Column height
L_b	Beam length
M_b	Flexural strength of beam section
M_c	Flexural strength of column section
n_t	Number of top bars in the beam
N_o^*	Minimum design axial column load at the ultimate limit state
P	Axial compression force on column
P_0	Lateral applied loading in Equation (9.2)

P_0	Nominal concentric compressive capacity of RC columns
P_y	Maximum story shear force
R	Radius of the beam reinforcement hook
R_d	Contribution of the diagonal mechanism to resist the joint shear force
R_h	Contribution of the horizontal mechanism to resist joint shear force
R_v	Contribution of the vertical mechanism to resist the joint shear force
s	Slip values in CEB-FIP model code
s	Spacing of shear reinforcement
s_c	Compression force in the central strut
s_i	Gauge length of the region i in beams
s_1, s_2, s_3	Slip values in CEB-FIP model code
s_v	Spacing of the joint hoops
t	Notch width in RILEM test
T	Stress in flexural tensile reinforcement
$T_{b,1}, T_{b,2}$	Tension force in the beam bar
u_b	Average bond stress
V	Total horizontal seismic force at the base of the structure
V_b	Beam shear force
V_c	Column shear force
V_{jh}	Horizontal joint shear force
V_{jx}	Horizontal joint shear force transferred in x direction

V_{jv}	Vertical joint shear force
V_{jz}	Horizontal joint shear force transferred in z direction
V_n	Joint shear strength
w_s	Half of the distance between the struts
w_u	Ultimate crack opening
$\alpha_1, \alpha_2, \alpha_3$	Coefficients defined in Eurocode 2
α_4, α_5	Coefficients defined in Eurocode 2
α_i	Factor for determining A_{jh}
α_0	Over-strength factor in Equation (6.4)
$\alpha_0, \alpha_f, \alpha_p$	Factor in Equation (6.3)
α_s, α_t	Factor in Equation (6.3)
α_n	Sectional curvature ductility of the plastic region
β	Factor in Equation (7.3)
β	Ratio of the compressive to tensile reinforcing beam bars
ε	Tensile strain in reinforcement
ε_0	Concrete cylinder strain corresponding to the concrete stress of f'_c
ε_{cr}^u	Ultimate strain in concrete
ε_{cx}	Average concrete strain in the x direction
ε_{cy}	Average concrete strain in the y direction
$\varepsilon_{uniaxial}^p$	Plastic strain in uniaxial stress direction
ε_y	Yield strain of steel reinforcement

ξ	Factor in Equation (7.1)
ξ	Softening coefficient
ξ_{eq}	Equivalent viscous damping
ρ_c	Ratio of total longitudinal reinforcement of column
ρ_s	Volumetric transverse reinforcement ratio
ρ_t'	Ratio of longitudinal high-strength steel reinforcement area divided by the total longitudinal reinforcement area
ρ_t	Ratio of total longitudinal reinforcement of column ($= A_{st}/A_g$)
η	Factor in Equation (7.1)
θ	Inclination of the principal compression
θ_1	Drift ratio at peak lateral resistance for positive loading for third cycle of loading sequence
θ_2	Drift ratio at peak lateral resistance for negative loading for third cycle of loading sequence
θ_1'	Drift ratio for zero lateral load for unloading at stiffness K from peak positive lateral resistance for third cycle of loading sequence
θ_2'	Drift ratio for zero lateral load for unloading at stiffness K' from peak negative lateral resistance for third cycle of loading sequence
$\theta_{b,i}$	Rotation measured over the region i of beam
$\theta_{b,fe}$	Beam fixed-end rotation
$\theta_{c,i}$	Rotation measured over the region i of column
$\theta_{c,fe}$	Column fixed-end rotation

ν_{cxy}	Average shear stresses of concrete in the x-y coordinate
τ_{av}	Average bond stress
τ_f	Residual bond stress in CEB-FIP model code
τ_{max}	Maximum bond stress in CEB-FIP model code
τ_u	Friction bond strength
$(\tau_{jh})_{cr}$	Shear stress values of the backbone key points
$(\tau_{jh})_y$	Shear stress values of the backbone key points
$(\tau_{jh})_{max}$	Shear stress values of the backbone key points
$(\tau_{jh})_{res}$	Shear stress values of the backbone key points
μ	Bond index factor
$\delta_{b,f}$	Flexural deformation of beam
$\delta_{b,fe}$	Fixed-end deformation of beam
$\delta_{c,f}$	Flexural deformation of column
$\delta_{c,fe}$	Fixed-end deformation of column
δ_{u1}	Factor in Newton-Raphson method
δ_{ua}, δ_{ub}	Ultimate displacement
δ_y	Yield drift ratio
${}_b\delta_i$	Bottom displacements measured over the region i in beam
${}_l\delta_i$	Left displacements measured over the region i in column
${}_l\delta_l$	Left displacements measured at the fixed-end interface

${}_r\delta_i$	Right displacements measured over the region i in column
${}_r\delta_l$	Right displacements measured at the fixed-end interface
${}_t\delta_i$	Top displacements measured over the region i in beam
$\phi_{b,i}$	Average curvature over the region i in beam
$\phi_{c,i}$	Average curvature over the region i in column
ϕ_y	Yield curvature
ζ	Factor in Equation (7.1)
Δ	Inter-story horizontal displacement
Δ_b	The horizontal displacement due to the beam deformations
Δ_c	The horizontal displacement due to the column deformations
$\Delta_{b,f}$	The equivalent horizontal displacement due to beam flexural deformation
$\Delta_{b,fe}$	The equivalent horizontal displacement due to beam fixed-end rotation
Δ_{u0}	Factor in Newton-Raphson method
Δ_{u1}	Factor in Newton-Raphson method
γ	Joint shear strength factor
γ_{cxy}	Average shear strain of concrete in the x-y plane
γ_j	Average shear distortion of the joint
γ_{cr}, γ_y	Shear strain values of the backbone key points
$\gamma_{\max}, \gamma_{res}$	Shear strain values of the backbone key points

CHAPTER 1

INTRODUCTION

1.1 Problem Statement

Before 1990, most of reinforced concrete (RC) structures were designed using steel reinforcement with yield strength of 420 MPa. Recently, the use of high-strength reinforcing bars has brought about numerous benefits to the construction industry including cost savings, reduced construction time, and reduction in reinforcement congestion. Utilizing high-strength steel (HSS) reinforcement proved to be effective in solving the steel congestion problem by reducing the required steel, especially in beam-column connections. The comparison of the stress-strain curves for normal-strength and high-strength steel reinforcement is as shown in **Figure 1.1**. A sharp yield plateau and minimum tensile-strength to yield-strength ratio of 1.25 are needed for the reinforcement to be used in seismic design applications.

The increased yield strength of longitudinal beam bars will result in higher development length and column depths, which are required for the bond action between the concrete and steel reinforcement. To address this issue, the use of high-strength concrete (HSC) coupled with HSS reinforcement has become widespread. In this study, HSC is defined as workable concrete with compressive strength greater than 60 MPa. The utilization of HSC in building structures offers various benefits, such as achieving greater heights by reducing the mass of required concrete, and reducing the size of columns. The smaller size of column cross sections leads to higher rentable floor space, which is beneficial from an architectural point of view. In addition, the high modulus of elasticity of the HSC will reduce deflections and creep deformations. High-strength concrete is also much more resistant to chemical deterioration because of its high cement content and lower value of water over cement ratio.

Figure 1.2 shows the relationship between the strength of the material and the research area and fields of development as presented by Li *et al.* [L3].

Figure 1.1 Stress-strain curves for normal and high-strength reinforcing steel

Figure 1.2 Relation of strength of material and fields of research and development [L3]

In the past three decades, the ‘A’ region has been the focal point of research, and the other regions of lower priority have been neglected. Consequently, the current design provisions in building codes such as ACI 318-11 [A1] and NZS 3101-06 [N3] may need to be revised as they are inadequate in considering high-strength material. With the increased utilization of HSC for different structural components, many have voiced concern over whether the current code provisions are applicable to the design of such members. Despite the popularity of using HSS reinforcement, the seismic provisions for special moment-resisting frames in ACI 318-11 [A1] set a limit of 420 MPa for longitudinal reinforcements and allow the yield strength of up to 689 MPa only for confining reinforcements. This restriction is due to the perceived lack of test data for cyclically loaded members with reinforcement grade of higher than 550 MPa. Hence, this study would mainly focus on the ‘B’ and ‘D’ regions.

Ehsani *et al.* [E2] presented the first reported study on the behavior of moment-resisting frames constructed with HSC. The effect of confining reinforcements and joint hoops in high-strength concrete members has been investigated by Li *et al.* [L3] and Hwang *et al.* [H7]. Some other test programs [A6, L2, L5] have been

reported to assess the bond condition in beam-column joints with Grade 500 longitudinal reinforcements, while Hwang *et al.* [H5] tested beam-column joints with Grade 600 longitudinal beam reinforcing bars. Nevertheless, the experimental evidence of the seismic behavior of beam-column joints with HSS longitudinal bars is still inadequate.

It is known that the increased bond-slip of reinforcing bars, decreased hysteretic energy dissipation, and lower shear strength of the joints are the key issues in designing beam-column joints utilizing HSS reinforcements. To prevent premature bond-slip in interior beam-column joints, some limitations on the beam-bar diameter to the column-depth ratio (d_b/h_c) have been suggested by NZS 3101-06 [N3]. However, the gap between the suggested requirements of various design codes increases as the yield strength of the reinforcing bars increases. Seven full-scale interior joints reinforced with HSS longitudinal bars with $d_b/h_c = 23.7$ and 28.1 were tested in this study. The d_b/h_c value was determined based on the ACI 318-11 [A1] requirement. However, the limitation of beam bar diameter suggested by NZS 3101-06 [N3] was not satisfied.

On the other hand, the critical issue when using HSS reinforcing bars in exterior joints is the increased anchorage length of the beam bars in the joint region. In the current study, a combination of HSC coupled with the use of HSS reinforcements has been introduced as a possible solution. ACI 318-11 [A1] has specified a minimum development length (l_{dh}) of a 90 degree hook bar as mentioned in **Equation 1.1a**. The compressive development length in the case of reversed cyclic loading in exterior joints is as mentioned in **Equation 1.1b**.

$$l_{dh} \geq f_y d_b / 5.4 \sqrt{f'_c} \text{ (MPa)} \quad (1.1a)$$

$$l_{dh} \geq f_y d_b / 4 \sqrt{f'_c} \text{ (MPa)} \quad (1.1b)$$

An analytical approach is used in the current study to take into account the effect of column compression loading in the above equations, and a novel equation is proposed. Five full-scale exterior joints utilizing HSS reinforcement with

$l_{dh}/d_b = 20.2$ and 24.2 were tested. The l_{dh}/d_b values satisfied the requirements of ACI 318-11 [A1] and ACI 352-02 [A3]. The applicability of ACI 318-11 [A1] provisions in the case of using HSS bars is to be verified. The originality of this study includes the experimental investigations on RC members with high-strength materials. The influence of column axial loading and other design parameters was studied through the developed numerical and analytical methods.

1.2 Seismic Design Philosophy of Reinforced Concrete Moment-Resisting Frames

There are two choices when designing an earthquake-resistant reinforced concrete structure. One is to design a structure with shear walls (braced frames) and the other is to design a ductile moment-resisting frame. Although shear walls in braced frames are very efficient in absorbing the earthquake energy and resisting lateral loads by providing adequate stiffness, the latter option is usually preferred as it provides greater flexibility for managing interior spaces, together with many other architectural advantages.

Structures designed according to building codes can withstand severe earthquake loading if they have the ability to dissipate significant seismic energy through inelastic action. This fact has resulted in prescriptive code provisions that limit the architectural advantages of the system, but makes the structural design more stable.

Figure 1.3 Types of beam-column joints [H1]

Beam-column joints in reinforced concrete structures as shown in **Figure 1.3** are key components indispensable to achieving the required integrity of the performance of building structures under seismic loadings. During seismic excitations, beam-column connections are subjected to severe shear stresses of the magnitudes much larger than those of the other frame members. If the demand on frames becomes larger than the capacity, the beam-column joint will become a weak link that dictates the strength and ductility of the frame. As exemplified in several past earthquakes such as El-Ansam (1980), Northridge (1994), Tehuacan (1999), Izmit (1999), Athens (1999), Chi-Chi Taiwan (1999), and Haiti (2010), the damage in beam-column joints was observed in a vast number of buildings, which resulted in partial or total collapse of the building. Since 1960, significant studies have been done to fulfill the seismic performance of reinforced concrete structures, with the beam-column joints being one of the most crucial components to study. These combined efforts have shed light on new design concepts to provide adequate strength and ductility to RC members and frames.

1.3 Objectives and Scopes of the Study

The aim of this research study is to provide experimental evidence of the seismic behavior of full-scale interior and exterior beam-column joints reinforced with HSS bars subjected to cyclic loading and with various design parameters. The test specimens were constructed using various grades of concrete to investigate the beneficial effect of using HSC coupled with HSS reinforcements. The joint shear behavior and bond strength of the specimens are further investigated and the validity of the design requirements of ACI 318-11 [A1] for HSS reinforcements is examined.

This research consists of the following experimental and analytical components:

- Conducting a series of experiments to study the seismic behavior of beam-column joints constructed using high-strength materials.
- Analyzing the test results and observations to understand the general behavior, cracking pattern, load-displacement responses, and failure mechanism of beams, columns, and joints.

- Modeling high-strength members and frames using the finite-element method and strut-and-tie approach.
- Undertaking a parametric study to provide additional information to understand the influence of various design parameters on seismic behavior of RC frame members.
- Evaluating the analysis results and proposing design equations for the detailing of reinforced concrete frames and members constructed using high-strength materials.

1.4 Organization of the Thesis

The thesis reports the research findings of experimental studies, finite element analysis, and parametric studies on seismic behavior of RC frame members and joints constructed using high-strength materials. It is organized into ten chapters as follows:

Chapter 1 introduces the background of this research project and the research significance. Based on the literature review, a database consisting of numerous test specimens of HSC beam-column joints, which were constructed with normal-strength and high-strength steel reinforcement, is compiled and processed in **Chapter 2**. In addition, the effect of various parameters on the behavior of joints is studied based on the available database.

Chapter 3 outlines the experimental program involving seven interior and five exterior full-scale beam-column joints. The specimens are designed and constructed using HSC and HSS longitudinal and transverse reinforcement. The variables in test specimens include the concrete compressive strength, the reinforcement yield strength and the level of axial compression loading. A typical quasi-static loading scheme is applied to all the test units in order to simulate lateral earthquake excitation.

In **Chapters 4 and 5**, the experimental result and observation including the cracking pattern, hysteretic behavior, lateral loading capacity, strain profiles of beam and column reinforcement, beam and column curvature distributions, and joint shear distortion and expansion are presented.

Further discussion of the test results is conducted in **Chapter 6**. The performance of test specimens is examined in terms of beam curvatures, secant stiffness, energy dissipation capacity, viscous damping, decomposition of lateral displacement, joint shear strength, bond performance of specimens, and the effect of varying axial loading on joints. In addition, a backbone curve model for the shear behavior of the joints is proposed. An explanation for the failure mechanism of specimens is also presented using the joint strength hierarchy assessment method.

Finite-element (FE) assumptions and modeling procedures are explained in **Chapter 7**. An extensive parametric study is performed using a nonlinear FE program, DIANA 9, to study the influence of various parameters on the strength, bond conditions, and energy dissipation capacity of the test specimens. All the models are calibrated and verified based on test results.

A reinforced concrete column design method, which aims to improve the ductile behavior of RC columns by utilizing various steel grades for longitudinal reinforcements, is evaluated in **Chapter 8**. Parametric studies via FE modeling are performed to study the influence of various design parameters on the ultimate drift capacity of RC columns. The correlation between the ultimate drift capacity and the confinement level, the axial load level, and the ratio and grade of HSS longitudinal reinforcement is investigated. Design charts for various ultimate drift levels in terms of other design parameters are also developed.

An explanation for the observed cracking pattern and further analytical investigation of the joint shear capacity of the specimens is conducted in **Chapter 9**. A contemporary bond distribution model is proposed based on experimental results and it is incorporated in an existing strut-and-tie model. The test specimens are studied using this novel strut-and-tie method and the results are compared to experimental observations.

Finally, the major conclusions from the study undertaken, and some suggestions for future research are given in **Chapter 10**.

Appendix I presents the database of beam-column joints constructed using high-strength materials. **Appendix II** illustrates the strain profile of steel reinforcement in test specimens, while **Appendix III** provides a code sample of FE modelling. **Appendix IV** summarises the calculation procedure of the strut-and-tie modelling.

CHAPTER 2

BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

The challenging issue in using high-strength steel (HSS) reinforcement in special moment frames is regarding the development length of the longitudinal bars in interior joints. Zhu and Jirsa [Z1] and Lin [L5] summarized the past research on the bond behavior and the required development lengths in interior joints. Zhu and Jirsa investigated the performance of a number of RC members which were constructed using reinforcement with yield strength of 420 MPa, with concrete strengths ranging from 27 to 35 MPa. Current recommendations in ACI 318-11 [A1] for the minimum joint depth of interior joints are based on the tests conducted by Zhu and Jirsa. These recommendations are presented in this chapter.

To supersede the reinforcement with yield strength of 420 MPa, Lin tested several beam-column joints incorporating reinforcement with yield strength of 517 MPa. The main variables in the tests were the joint reinforcement ratio, the axial load level, and the ratio of top and bottom beam reinforcing bars. Based on the test results, Lin recommended an equation to calculate the minimum joint depth in interior joints. This equation is incorporated in the reinforced concrete design approach in the New Zealand NZS 3101-06 [N3] code.

In addition, Aoyama [A6] reported the test results of an experimental program on the bond strength of reinforcement with the yield strength of 420 to 690 MPa. Recommendations on the minimum joint depth in moment frames in Japan were developed based on the tests conducted by Aoyama.

In this chapter, a literature review is presented regarding the conducted research and design methods related to the construction of beam-column joints with high-strength materials. Previous experimental research and their subsequent conclusions are mentioned in Section 2.2. In addition, the analytical models for the joint shear prediction are described in Section 2.3. Furthermore, a database is developed in Section 2.4 based on the past experiments reported in the literature. The database is used to explore the influence of the key parameters on seismic performance of

interior and exterior beam-column joints constructed using high-strength materials. Finally, the code provisions with regards to the design of beam-column joints recommended by ACI 318-11 [A1] and NZS 3101-06 [N3] are presented in Section 2.5.

2.2 Recent Experimental Investigations on HSC Beam-Column Joints

2.2.1 Research conducted by Ehsani *et al.* (1987)

The first reported study on the behavior of moment-resisting frames constructed with high-strength concrete (HSC) was done by Ehsani [E2]. Four exterior reinforced concrete beam-column subassemblies constructed using HSC was tested and the results were compared to similar specimens constructed with normal-strength concrete. Grade 60 reinforcing steel with the yield strength varying from 428 to 483 MPa was used for Specimens 1 through 4. In Specimen 5, Grade 40 steel with $f_y = 275\text{MPa}$ was used for the longitudinal beam reinforcement and Grade 60 steel was used for the column and joint reinforcement. The concrete compressive strength was 66 MPa in all specimens.

According to the obtained hysteresis response, the HSC specimens were able to dissipate larger amount of energy. In addition, the stiffness of the specimens decreased as the number of loading cycles increased. This is as a result of the concrete degradation within the joint region as well as the slippage of the longitudinal bars in beams or columns. The observed pinching behavior in hysteresis loops was because of the inability of the formed cracks at the column face to close.

In general, two types of failure modes were observed. Flexural hinges were developed at the beam ends facing the column in Specimens 1, 2, 3, and 5. On the other hand, joint shear failure was observed in Specimen 4 as a result of the high joint shear stress. The column flexural strength in Specimen 4 was 67% higher than the beam flexural strength.

The joint shear stress in specimens varied from 5.03 to 8.76 MPa and it had a distinct effect on the failure mode of the specimens. Specimens 1 and 2 had a low

joint shear stress and they failed because of the formation of the beam plastic hinge. On the other hand, Specimen 4 with the largest joint shear stress failed in shear. In Specimens 3 and 5 with moderately-high joint shear stress values, plastic hinges were formed at the beam end. In conclusion, test results indicated that:

1. Beam-column joints constructed using HSC exhibited ductile hysteretic response, similar to that of ordinary-strength concrete joints.
2. For corner joints constructed with Grade 66 concrete, the maximum allowable joint shear stress was $(0.87\sqrt{f'_c} \text{ MPa})$.
3. The mode of failure is determined based on the combination of flexural strength ratio and joint shear stress, rather than by the flexural strength ratio alone.
4. The energy dissipation capacity of the joints is reduced in case of high joint shear stress values.
5. Buckling of compression longitudinal bars is more severe in the case of lower joint shear stress values.

2.2.2 Research conducted by Alameddine (1990)

Twelve HSC exterior beam-column joints were tested by Alameddine [A5]. Beams and columns were extending from the joint to the inflection points, which were the mid-span points for the beam, and the mid-height points of the upper and lower stories for the columns. The range of concrete compressive strength in specimens was from 55 to 96 MPa.

All the specimens were designed according to ACI 318 provisions. The dimension of beams and columns was similar in all of the specimens and the investigating parameters included the concrete compressive strength and the amount of confinement steel reinforcement. The following conclusions were drawn based on the study:

1. The behavior of the properly detailed HSC joints was more ductile as compared to the ordinary-strength concrete specimens.

2. Increasing the ratio of confining reinforcement improved the behavior of joints and helped to delay or eliminate the bar pullout and the slippage of longitudinal column bars.
3. The energy dissipation capacity of the specimens with high joint shear stress values was less. In all of the specimens with high joint shear stress, there was no yielding of the longitudinal beam bars and an early failure of joints was observed.
4. Some suggestions were made to modify the ACI 352 provisions for HSC frames.

2.2.3 Research conducted by Sugano *et al.* (1990)

Simulated earthquake loading test on eight half-scale interior beam-column joints were performed by Sugano *et al.* [S6]. The specimens were constructed using different concrete compressive strengths of 39, 59 and 78 MPa. The yield strength of longitudinal reinforcing bars varied from 385 to 568 MPa, while the transverse reinforcement in beams and columns were of 900 MPa steel. The main test variables were the concrete compressive strength, joint shear stress and the joint detailing.

Specimens with the concrete compressive strength of 39 MPa failed in shear before any beam flexural yielding. These specimens showed a considerable deterioration in load carrying capacity during the test. Other specimens formed beam plastic hinges and showed a stable hysteresis behavior up to a DR of 5%, and they failed in joint shear at drift ratios larger than 5%. The specimens with HSS beam bars exhibited flexural beam yielding. The story shear versus story drift was stable up to the drift ratio of 5% and no significant pinching effect was observed. However, these specimens with HSS reinforcement experienced more severe damage in the joint region because of the higher strength of beam bars and higher joint shear stress values. Sugano made the following key conclusions based on the test observations:

1. The use of HSS reinforcement in columns can effectively confine the concrete with the compressive strength of up to 78 MPa.

2. To ensure the ductile behavior of HSC columns, the full capacity of transverse reinforcement normalized by the concrete compressive strength should be constant at a level of 0.1 or greater.
3. Under high joint shear stress, HSC interior joints with the concrete compressive strength of larger than 59 MPa had a sufficient load carrying capacity up to a drift ratio of 5%.
4. To prevent a premature joint shear failure, the nominal joint shear stress should be less than $5.0\sqrt{f'_c}$ for concrete compressive strengths of 35 to 88 MPa.

2.2.4 Research conducted by Ehsani and Alameddine (1991)

Ehsani [E1] studied the accuracy of the equation suggested by ACI 318 for the prediction of joint shear stress ($\gamma\sqrt{f'_c}$) in HSC corner beam-column joints. Twelve specimens constructed using HSC with compressive strength of 55.8, 73.8 and 93.8 MPa and Grade 60 reinforcing steel with actual yield strength ranging from 421 to 496 MPa were tested. The studied variables in this experimental program included the concrete compressive strength, the joint shear stress which ranged from 7.6 to 9.7 MPa, and the joint confinement ratio. Stiffness reduction was observed during subsequent displacement cycles for all of the specimens. This reduction was mainly because of the concrete deterioration in and adjacent to core region and also the slippage of the longitudinal bars in beams and columns. Only the specimens with low joint shear stress and high joint confinement could achieve the expected strength of the beam.

According to the test results, Ehsani proposed the following model to determine the maximum allowable joint shear stress for normal-strength and high-strength concrete specimens:

$$v_j = \frac{\gamma}{12} \times \sqrt{f'_c}, \quad f'_c \leq 41.3 \text{ MPa} \quad (2.1a)$$

$$v_j = \frac{\gamma}{12} \times \sqrt{41.3} + 0.43 \times \sqrt[3]{f'_c - 41.3}, \quad f'_c \geq 41.3 \text{ MPa} \quad (2.1b)$$

It was concluded that the proposed joint shear stress is not related to the factor γ . However, the joint shear stress ($\gamma\sqrt{f'_c}$) depends on the joint type and the geometric classification. Therefore, the authors suggested that the proposed model for HSC specimens could also be used for all of the joint types and classifications.

2.2.5 Research conducted by Fujii (1991)

Eight one-third scale specimens including four interior and four exterior beam-column joints were tested by Fujii and Morita [F2]. The test variables included the yield strength of beam reinforcing bars, the column axial loading level, and the ratio of joint hoops. Utilizing HSS reinforcement, the program was determined in a way that the joint shear failure occurs prior to beam or column yielding in all of the specimens.

Constant axial compression loading was applied on top of the column, while a reversed cyclic horizontal loading pattern was applied to each beam tip in the opposite direction. The typical cracking pattern of the specimens is shown in **Figure 2.1**. According to experimental observations, Fujii and Morita [F2]

concluded that the increase of column axial loading from $\frac{f'_c}{12}$ to $\frac{f'_c}{4}$ had no influence on the ultimate shear strength of the interior joints, while it increased the shear strength of the exterior joints around 11%. It was observed that the increase of transverse reinforcement ratio from 0.41% to 1.1% did not have any notable influence on the behavior of both types of the testing joints.

2.2.6 Research conducted by Kitayama *et al.* (1992)

In order to study the bond transfer mechanism along the beam reinforcement and the shear resistance of joints, five half-scale RC interior beam-column joints were tested by Kitayama *et al.* [K1]. The dimension of beams was 200×300 mm and the column dimension was 300×300 mm in all of the specimens. Specimen I3 was constructed using ordinary-strength material. In other specimens, the yield strength of reinforcing bars varied from 770 to 800 MPa, while the concrete compressive strength was between 85 to 99 MPa. The input joint shear stress was restricted to $0.15f'_c$ and the columns were designed to remain elastic during the test.

A bidirectional loading pattern was applied to Specimen I2, while a unidirectional loading was implemented for other specimens. Constant column axial loading was present all the time during the test. Final cracking pattern of the specimens is shown in **Figure 2.2**.

Based on the test results, the local bond stress along beam bars in the joint region is shown in **Figure 2.3**. In Specimen I5, the local bond stress (τ_1) increased as the story drift increased and a good bond was maintained during the test. In Specimen I6 with larger bar diameters, the value of (τ_1) increased until DR=2.7%, and a significant decay was observed in following drift ratios due to the bond deterioration. In Specimen I3 which was constructed using normal-strength materials, the bond stress values were smaller than those of the two HSC specimens. However, the bond stress values normalized by the square root of concrete compressive strengths were the same for Specimens I3 and I6.

An average bond stress (U_b) along the beam tensile and compressive bars is expressed as follows:

$$U_b = (1 + \beta) \left(\frac{f_y}{4} \right) \left(\frac{d_b}{h_c} \right) \quad (2.2)$$

where:

β is the ratio of the compressive to tensile reinforcing bar area;

f_y is the yield strength of the beam bar;

d_b is the beam bar diameter;

h_c is the column depth.

Figure 2.1 Typical crack pattern of the specimens tested by Fujii and Morita [F2]

(a) Specimen I1

(b) Specimen I5

(c) Specimen I2 (east-west direction)

(d) Specimen I2 (south-north direction)

(e) Specimen I6

Figure 2.2 Final crack pattern of the specimens tested by Kitayama *et al.* [K1]

According to the tests conducted at the University of Tokyo, the measured average bond stress (τ_{av}) is related to U_b as shown in **Figure 2.4**. As illustrated, deterioration points indicate the cases in which the average bond stress (τ_{av}) has deteriorated in the joint, either before or after the yielding of beam bars. On the other hand, good bond points represent the specimens in which the bond behavior has been satisfactory, even after the beam bar yielding. According to these test results, the average bond stress (τ_{av}) does not decay before the beam yielding if its value is controlled by $0.42U_b$ and $0.5U_b$ for the top and bottom reinforcements, respectively.

Furthermore, the relationship between the average bond stress and the concrete compressive strength can be shown as **Figure 2.5**. It is concluded that the average bond stress (τ_{av}) along the top and bottom bars is estimated as $0.084f'_c$ and $0.126f'_c$, respectively. It was observed that the bond deterioration in a joint region will not occur if the bond index (U_b/f'_c) is smaller than 0.2 for the top and 0.25 for the bottom reinforcing bars.

According to the tests on HSC interior beam-column joints, Kitayama *et al.* [K1] concluded that the bond strength along longitudinal beam bars was not affected by the bar diameter and remained satisfactory up to a story drift ratio of 4%. In addition, it was observed that the bidirectional loading reversals did not have much influence on the bond transfer along beam bars.

Figure 2.3 Local bond stress within the joint for the specimens tested by Kitayama *et al.* [K1]

Figure 2.4 Measured bond stress relations in the specimens tested by Kitayama *et al.* [K1]

Figure 2.5 Relationship between the bond stress and concrete compressive strength in the specimens tested by Kitayama *et al.* [K1]

2.2.7 Research conducted by Oka and Shiohara (1992)

To evaluate the availability of design provisions for HSC interior beam-column joints in high seismic regions, Oka and Shiohara [O1] tested eleven 1/2.5 scale specimens constructed using HSC with normal or high-strength steel reinforcement. High-strength deformed bars of D13 and D19 with minimum yield strength of 500 MPa were used in the test. In addition, five sets of HSS hoops of D6 were used in the joint area to confine the joint core.

Test results showed that the longitudinal beam bars yielded approximately at a story drift ratio of 2%. The story drift at which the beam bars started to yield was increased as higher-strength longitudinal beam bars were used. In addition, it was observed that utilizing HSC did not change the crack pattern of the specimens.

Specimens J-1 and J-7 showed slight strength degradation until the end of the test, while Specimen J-10 showed sudden strength degradation because of the joint shear failure prior to beam yielding. The hysteresis response of Specimen J-9, which was constructed with the slab and transverse beams, showed a high energy dissipation capacity and no strength degradation was observed.

Oka and Shiohara concluded that ACI 318 recommendations showed a good correlation with the test results of HSC specimens. **Figure 2.6** shows the test results of the specimens with various concrete compressive strengths. The vertical axis shows the maximum joint shear stress, and the ACI 318 recommendation has been illustrated for the shear stress of the confined joints. A correlation can be observed between the code recommendation and the test results. It was observed that the maximum joint shear was influenced by the amount of longitudinal reinforcement. However, it was not specified whether the results were affected by the beam or column reinforcement. As the longitudinal reinforcement ratio increased, the observed joint shear stress increased up to 10%.

2.2.8 Research conducted by Noguchi and Kashiwazaki (1992)

An experimental study was conducted by Noguchi and Kashiwazaki [N2] to investigate the shear performance of beam-column joints. Six one-third scale interior joints constructed using HSC and HSS reinforcement were tested. At the end of the test, the opening of joint shear cracks and splitting cracks along the longitudinal column bars was remarkable in all of the specimens. The pinching effect, which was caused due to the bond deterioration of longitudinal beam bars through the joint, was not observed in hysteresis loops of the test specimens.

The relationship between the concrete compressive strength and the joint shear stress, which was obtained based on the test data for the specimens constructed with normal-strength and high-strength materials, is shown in **Figure 2.7**. It is observed that the maximum joint shear can fit accurately to the curve of $1.88\sqrt{f'_c}$.

The main findings of this study are summarized as follows:

1. The maximum joint shear strength for normal-strength and HSC specimens is nearly located on the curve of $1.88\sqrt{f'_c}$.
2. It was possible to restrain the bond deterioration along the longitudinal beam bars until the story drift angle of 1/100 rad.
3. The loading history had a great effect on the hysteresis characteristics after the maximum strength. The strength decay in specimens under reversed cyclic loading was not observed in the specimens under monotonic loading.
4. The role of lateral reinforcement and their confinement effect to the joint core became remarkable in large deformation levels after the drift angle of 1/150 rad.

2.2.9 Research conducted by Hwang *et al.* (2005)

A comprehensive study was conducted on the effect of joint hoops on the shear strength of exterior beam-column joints by Hwang *et al.* [H7]. Nine exterior joints with Grade 70 MPa concrete and Grade 420 MPa steel reinforcement were tested. A

displacement-controlled cyclic horizontal loading was applied to the beam end. The test setup is shown in **Figure 2.8**.

Although the specimens designed without joint hoops failed in joint shear, the hysteretic response was satisfactory and a ductility factor of 5.2 was reached. This indicates that designing the joints with sufficient shear strength, even without shear reinforcement, ensures a certain level of joint performance.

Based on test results of these nine exterior beam column joints, the following conclusions were made:

1. The current ACI 318 requirements for the amount of joint hoops are not necessary. Test results showed that the less number of joint hoops with the spacing of up to 300 mm can be used without any major difference in the joint performance.
2. Without the presence of column axial loading, beam-column joints could perform satisfactorily even if they are detailed without joint hoops. However, the adequate shear strength should be provided according to the strut-and-tie model.
3. The deterioration of joints under loading reversals can be restrained if the joint hoops are designed in a way to remain elastic during the loading. It is recommended to provide adequate number of hoops and with sufficient shear strength to make sure that the hoops will remain elastic during displacement reversals.

Figure 2.6 Relationship between the maximum joint shear and the concrete compressive strength in the specimens tested by Oka and Shiohara [O1]

Figure 2.7 Joint shear stress versus concrete compressive strength in the specimens tested by Noguchi and Kashiwazaki [N2]

Figure 2.8 Test setup and configuration of the specimens tested by Hwang *et al.* [H7]

2.2.10 Research conducted by Brook *et al.* (2006)

Brook *et al.* [B4] conducted a study on RC joints to evaluate the accuracy of bond strength models in NZS 3101-06 [N3]. Four beam-column joints were constructed using high-strength and large-diameter reinforcing bars. Drift-based cyclic loading was applied to the specimens up to a DR of 5%. Although all the four specimens were cycled to drift levels considerably higher than NZS 3101-06 [N3] drift limits, the bond failure happened only in two of them. It was concluded that the satisfactory bond performance was due to the large excesses of column moment capacity and vertical joint shear capacities. In addition, no relationship was found between the horizontal joint shear reinforcement and the bond performance.

According to the test outcome, the following conclusions were drawn:

1. The performance of the joints with large diameter and HSS reinforcement was satisfactory at drift levels corresponding to NZS 3101-06 [N3] limits. This indicates that current NZS 3101-06 [N3] design rules are conservative.
2. Only two of the four specimens experienced bond failure. This was in spite of loading all the specimens to drift levels significantly higher than those specified by NZS 3101-06 [N3].
3. The better-than-expected bond strength of the specimens was due to the large quantity of longitudinal column reinforcement. This reinforcement caused the columns to have a larger moment and vertical joint shear capacity. As a result, the damage was reduced in the joint panel and the bond condition improved.
4. Test results suggested that large quantity of vertical joint shear reinforcement improves the bond characteristics in the joint zone. In addition, no relationship was found between the bond performance and the horizontal joint shear reinforcement.

2.2.11 Research conducted by Hwang *et al.* (2014)

An experimental study was conducted by Hwang *et al.* [H5] to evaluate the seismic behavior of beam-column joints using Grade 600 MPa longitudinal bars in the beam. Four full-scale interior and three exterior beam-column joints were designed according to the special seismic provisions of ACI 318-11 [A1]. The beam-bar

diameter to column-depth ratios was 1/20.5 and 1/25.0 in specimens. Following conclusions were drawn based on the experimental observations:

1. The damage mode was diagonal cracking at the joint, yielding of beam flexural bars, and bond-slip of the beam bars.
2. The load-carrying capacities of the specimens agree with the predicted values.
3. The specimens in which the requirement of bond development length was not satisfied, the bond-slip of longitudinal beam bars was observed and the drift capacity of the specimens was decreased.
4. The joint shear strength specified in ACI 318-11 [A1] is adequate. However, the recommendations of the required development length of 90-degree hook bars in ACI 318-11 [A1] need to be revised.
5. Due to the lack of experimental data, authors suggested that further studies are required to confirm the test results.

2.3 Analytical Investigations on Shear Behavior of Beam-Column Joints

2.3.1 Strut-and-tie model proposed by Parker and Bullman (1997)

Parker and Bullman [P3] developed a strut-and-tie shear strength model. The authors intended to develop a model to be applicable to shear critical structural members such as beam-column joints. The generalized strut-and-tie model is illustrated in **Figure 2.9**. The shear span is defined empirically as below:

$$a_v = 0.8d_b - 0.8R \quad (2.3)$$

Where d_b is the effective depth of the beam and R is the radius of the beam reinforcement hook, which is positive for bars bent down into the joint and negative for bent up bars.

The critical inclination of the strut can be found using the following equations:

$$\tan \theta_{crt} = 1 - \frac{\gamma}{2}; \text{ for } \gamma \leq 0.5 \quad (2.4a)$$

$$\tan \theta_{crt} = \frac{1}{2\gamma} + \frac{\gamma^2 - 0.75}{6\gamma^3 + 2.5\gamma}; \text{ for } \gamma > 0.5 \quad (2.4b)$$

Where γ is defined as $\gamma = a_v/d$

Figure 2.9 Generalized monotonic STM proposed by Parker and Bullman [P3]

2.3.1.1 Beam-column joints without links

The shear strength may be controlled by the strength of the longitudinal steel bars as:

$$V_1 = (A_{sc} f_y + N) \tan \theta_{crt} \quad (2.5)$$

Where A_{sc} is the total area of the column reinforcement.

On the other hand, the shear strength of the joint may be limited by the strength of the concrete strut as:

$$V_2 = \alpha v f_{cu} b d \quad (2.6)$$

Where v is the effectiveness factor which is equal to $0.56 - \frac{f_{cu}}{310} \geq 0.4$ and the factor α is as defined below:

$$\alpha = \frac{1 - \tan \theta_{crt}}{\tan \theta_{crt} + 1/\tan \theta_{crt}} \quad (2.7)$$

2.3.1.2 Beam-column joints with links

The shear strength may be limited by the strength of the links as:

$$V_3 = V_2 + A_{sv} f_{yv} \left(\frac{a_v}{s_v} - 1 \right) \quad (2.8)$$

Where:

A_{sv} is the area of links

f_{yv} is the yield strength of links

a_v is the shear span

s_v is the links spacing

The shear strength cannot go beyond the strength of the concrete strut which is defined as:

$$V_4 = \beta V_2 \quad (2.9)$$

Where:

$$\beta = \frac{0.9d - s_v \tan \theta}{d - a_v \tan \theta} \geq 1 \quad (2.10)$$

2.3.2 Strut-and-tie model proposed by Hwang and Lee (2000)

Hwang and Lee [H6] proposed a softened strut-and-tie model for interior and exterior joints. In order to develop the model, three force transfer mechanisms were considered. The diagonal, horizontal and vertical mechanisms are presented in **Figure 2.11**.

The SSTM is based on the strut-and-tie concept and satisfies the three fundamental rules of mechanics including the stress equilibrium, strain compatibility, and the constitutive relationship of materials. The strut-and-tie model for the load transfer in a beam-column joint is illustrated in **Figure 2.12**. The force is transferred via a statically indeterminate load path. The angle of inclination of the single strut in the diagonal mechanism is defined as:

$$\theta = \tan^{-1} \left(\frac{h_b''}{h_c''} \right) \quad (2.11)$$

where h_b'' and h_c'' are the distances between extreme longitudinal steel bars in beams and columns. The area of the diagonal strut A_{str} is calculated as below:

$$A_{str} = a_s \times b_s \quad (2.12)$$

where a_s and b_s are the depth and width of the diagonal strut. b_s can be determined according to the ACI 318-11 [A1] definition of effective joint width. In addition, a_s can be approximated by the depth of column compression zone (a_c). This approximation will limit the accuracy of this model to the cases where the beam yielding is happening earlier than the joint failure.

$$a_c = (0.25 + 0.85 \frac{P}{A_g f_c'}) h_c \quad (2.13)$$

Figure 2.10 STM for beam-column joints with links proposed by Parker and Bullman [P3]

Figure 2.11 SSTM force transfer mechanisms proposed by Hwang and Lee [H6]

Figure 2.12 STM in beam-column joints proposed by Hwang and Lee [H6]

Where

P is the column axial load;

f'_c is the compressive strength of the standard concrete cylinder;

A_g is the gross area of the column section.

Satisfying equilibrium equations, the horizontal joint shear V_{jh} and the vertical joint force V_{jv} are determined as:

$$V_{jh} = -D \cos \theta + F_h + F_v \cot \theta \quad (2.14)$$

$$V_{jv} = -D \sin \theta + F_h \tan \theta + F_v \quad (2.14b)$$

Where:

D is the compression force in the diagonal strut;

F_h is the tension force in the horizontal tie;

F_v is the tension force in the vertical tie.

The forces in diagonal strut and horizontal and vertical ties can be stated as:

$$D = \frac{1}{\cos \theta} \cdot \frac{R_d}{R_d + R_h + R_v} V_{jh}, \quad R_d = \frac{(1 - \gamma_h)(1 - \gamma_v)}{1 - \gamma_h \gamma_v} \quad (2.15)$$

$$F_h = \frac{R_h}{R_d + R_h + R_v} V_{jh}, \quad R_h = \frac{\gamma_h(1 - \gamma_v)}{1 - \gamma_h \gamma_v} \quad (2.15b)$$

$$F_v = \frac{1}{\cot \theta} \cdot \frac{R_v}{R_d + R_h + R_v} V_{jh}, \quad R_v = \frac{\gamma_h(1 - \gamma_h)}{1 - \gamma_h \gamma_v} \quad (2.15c)$$

Where R_d , R_h and R_v are the contribution ratios of the diagonal, horizontal and vertical mechanisms to resist the joint shear force.

In order to investigate whether the joint strength is reached, the maximum compressive stress acting on the nodal zone is defined as below:

$$\sigma_{d,\max} = \frac{1}{A_{str}} \left(D + \frac{\cos \left(\theta - \tan^{-1} \left(\frac{h_b''}{2h_c''} \right) \right)}{\cos \left(\tan^{-1} \left(\frac{h_b''}{2h_c''} \right) \right)} F_h + \frac{\cos \left(\tan^{-1} \left(\frac{2h_b''}{h_c''} \right) - \theta \right)}{\sin \left(\tan^{-1} \left(\frac{2h_b''}{h_c''} \right) \right)} F_v \right) \quad (2.16)$$

The ascending part of the constitutive relationship for cracked concrete is formulated as:

$$\sigma_d = \xi \cdot f_c' \left(2 \left(\frac{\varepsilon_d}{\xi \varepsilon_0} \right) - \left(\frac{\varepsilon_d}{\xi \varepsilon_0} \right)^2 \right) \quad \text{for } \frac{\varepsilon_d}{\xi \varepsilon_0} \leq 1 \quad (2.17)$$

Where ξ is the softening coefficient:

$$\xi = \frac{5.8}{\sqrt{f'_c}} \cdot \frac{1}{\sqrt{1 + 400\varepsilon_r}} \leq \frac{0.9}{\sqrt{1 + 400\varepsilon_r}} \quad (2.18)$$

ε_0 is the concrete cylinder strain corresponding to the concrete stress of f'_c , which can be evaluated as:

$$\varepsilon_0 = -0.002 - 0.001 \left(\frac{f'_c - 20}{80} \right) \quad \text{for } 20 \leq f'_c \leq 100 \text{ MPa} \quad (2.19)$$

The constitutive material models are presented in **Figure 2.13**.

Figure 2.13 Constitutive models for concrete and steel reinforcement [H6]

The shear strength of the joint is assumed to be reached in the following concrete compressive stress and strain situation:

$$\sigma_d = \xi \cdot f'_c \quad (2.20)$$

$$\varepsilon_d = \xi \cdot \varepsilon_0 \quad (2.21)$$

For the joint hoops, stress-strain relationship of the steel is assumed to be in an elastic-perfectly-plastic state:

$$f_s = E_s \cdot \varepsilon_s ; \text{ for } \varepsilon_s < \varepsilon_y \quad (2.22)$$

$$f_s = f_y ; \text{ for } \varepsilon_s \geq \varepsilon_y \quad (2.22b)$$

Thus, the tension forces in the horizontal and vertical ties are:

$$F_h = A_{th} E_s \varepsilon_h \leq F_{yh} \quad (2.23)$$

$$F_v = A_{tv} E_s \varepsilon_v \leq F_{yv} \quad (2.23b)$$

Based on the two-dimensional compatibility condition as shown in **Figure 2.14** and considering the predefined angle of the principal compressive stress θ , the principal tensile strain ε_r can be expressed in terms of the vertical, horizontal and principal compressive strains as follows:

$$\varepsilon_r = \varepsilon_h + (\varepsilon_h - \varepsilon_d) \cot^2 \theta \quad (2.24)$$

$$\varepsilon_r = \varepsilon_v + (\varepsilon_v - \varepsilon_d) \tan^2 \theta \quad (2.24b)$$

Hwang and Lee [H6] categorized the solution procedure into five types according to the different yielding condition of the ties. The proposed model was used to determine the joint shear of different test specimens in literature. This experimental verification showed that the model is capable of predicting the shear force in both interior and exterior beam-column joints.

Figure 2.14 2D compatibility conditions in the study of Hwang and Lee [H6]

2.3.3 Analytical model proposed by Wong (2005)

Wong [W1] developed a new model to overcome the shortenings of the previous models such as modified compression field theory (MCFT), rotating angle softened truss model (RA-STM), and fixed angle softened truss model. This model satisfies stress equilibrium, constitutive laws, and compatibility conditions. The average stresses of cracked concrete are defined as:

$$f_{cx} = f_{c1} - \nu_{cxy} \cot \theta \quad (2.25)$$

$$f_{cy} = f_{c1} - \nu_{cxy} \tan \theta \quad (2.25b)$$

$$\nu_{cxy} = \frac{f_{c1} - f_{c2}}{2} \sin 2\theta \quad (2.25c)$$

Where

f_{cx} and f_{cy} are the average concrete stresses in x and y directions;

f_{c1} and f_{c2} are the average principal stresses of concrete in 1 and 2 directions;

ν_{cxy} is the average shear stress of concrete in the x-y coordinates;

θ is the inclination of principal compression.

The corresponding average strains of the cracked concrete are as below:

$$\varepsilon_{cx} = \frac{\varepsilon_{c1} - \varepsilon_{c2}}{2} (1 - \cos 2\theta) + \varepsilon_{c2} \quad (2.26)$$

$$\varepsilon_{cy} = \varepsilon_{cx} + (\varepsilon_{c1} - \varepsilon_{c2}) \cos 2\theta \quad (2.26b)$$

$$\gamma_{cxy} = 2(\varepsilon_{c1} - \varepsilon_{c2}) \tan \theta \quad (2.26c)$$

Where

ε_{cx} and ε_{cy} are the average concrete strains in x and y directions

ε_{c1} and ε_{c2} are the average principal strains in 1 and 2 directions

γ_{cxy} is the average shear strain of concrete in x-y plane.

The concrete constitutive law adopted in this model is defined as follows:

$$f_{c2} = \xi \cdot f_c' \left(2 \left(\frac{\varepsilon_{c2}}{\xi \varepsilon_0} \right) - \left(\frac{\varepsilon_{c2}}{\xi \varepsilon_0} \right)^2 \right); \text{ for } \frac{\varepsilon_{c2}}{\xi \varepsilon_0} \leq 1 \quad (2.27)$$

$$f_{c2} = \xi \cdot f_c' \left(1 - \left(\frac{\frac{\varepsilon_{c2}}{\xi \varepsilon_0} - 1}{\frac{4}{\xi} - 1} \right)^2 \right); \text{ for } \frac{\varepsilon_{c2}}{\xi \varepsilon_0} > 1 \quad (2.27b)$$

Where ξ is the softening coefficient as defined below:

$$\xi = \frac{5.8}{\sqrt{f_c'}} \cdot \frac{1}{\sqrt{1 + 400 \varepsilon_{c1}}} \leq \frac{0.9}{\sqrt{1 + 400 \varepsilon_{c1}}} \quad (2.28)$$

The average concrete tensile stress-strain relationship can be shown as:

$$f_{c1} = E_c \varepsilon_{c1}; \text{ for } \varepsilon_{c1} \leq \varepsilon_{cr} \quad (2.29)$$

$$f_{c1} = f_{cr} \left(\frac{\varepsilon_{cr}}{\varepsilon_{c1}} \right)^{0.4}; \text{ for } \varepsilon_{c1} > \varepsilon_{cr} \quad (2.29b)$$

Where

$$E_c = 3875 \sqrt{f_c'} \quad (2.30)$$

$$\varepsilon_{cr} = 0.8 \times 10^{-4} \quad (2.30b)$$

$$f_{cr} = 0.31 \sqrt{f_c'} \quad (2.30c)$$

There are some algorithms presented in order to find the shear strength of exterior beam-column joints using the above mentioned formulas.

2.4 The Influence of Various Design Parameters on Seismic Behavior of Beam-Column Joints

This section explores the effect of various design parameters influencing the seismic performance of interior and exterior beam-column joints which are constructed using high-strength materials. The database is developed based on the experimental data available in the literature and further investigations are conducted on this database to determine the influence of various design parameters. The investigated parameters included the concrete compressive strength, reinforcement yield strength, axial load level and the failure mode of specimens.

2.4.1 Definition of joint shear strength

In order to compare the joint shear of different specimens, the measured joint shear is normalized as follows:

$$\gamma_j = \frac{V_j}{A_j \sqrt{f'_c}} \quad (2.31)$$

Where

V_j is the maximum joint shear;

A_j is the effective joint shear area as defined in ACI 352-02 [A3];

f'_c is the concrete compressive strength.

By taking the column shear force V_{col} acting on the top of the column and the plastic hinges forming in beams at the column face, the shear force within interior joints can be evaluated using the following equilibrium equation:

$$V_j = T_b + C_c + C_s - V_{col} \quad (2.32)$$

The tension force of the bottom longitudinal beam bar is taken as T_b , which is defined as the yield strength of reinforcing bar (f_y) multiplied by the over-strength

factor (α). On the opposite face, the steel and concrete are in compression and their forces are shown as C_s and C_c .

In the case of exterior beam-column joints, the maximum joint shear can be related to the beam shear force as presented in the following equation:

$$V_j = T_b - V_{col} \quad (2.33)$$

Where T_b is the tension force in the beam reinforcement, which can be estimated using the following equation:

$$T_b = \frac{M_b}{jd_b} \quad (2.34)$$

Where j is the lever arm and d_b is the effective beam depth. The lever arm could be calculated using the section analysis method or could be estimated as 0.9 for BJ-failure and 0.875 for J-failure specimens. The beam moment is calculated using the following equation:

$$M_b = V_b L_b \quad (2.35)$$

Where L_b is the beam length as illustrated in **Figure 2.15**.

Figure 2.15 Statics of a beam-column joint subassembly [H1]

2.4.2 Definition of ultimate displacement

In some cases in the literature, the displacement of test specimens is normalized with respect to the yield displacement. Hence, the ductility factor is defined as the ratio of ultimate displacement over the yield displacement. There are two types of approach in order to find the ultimate displacement.

In the first approach, the ultimate displacement is defined as the displacement corresponding to the maximum lateral load resistance (δ_{ua}). The second approach defines the ultimate displacement (δ_{ub}) as the displacement corresponding to the lateral load equal to the 85% of the maximum lateral loading resistance.

To obtain the yield displacement, a secant is drawn at the point of 0.75 of the maximum resistance. The intersection of the secant with the horizontal line at the peak resistance will define the yield displacement. **Figures 2.16** and **2.17** show different approaches to find the ultimate and yield displacement. The displacement ductility is estimated as follows:

$$\mu_{\delta} = \frac{\delta_u}{\delta_y} \quad (2.36)$$

Where δ_u and δ_y are the ultimate and yield drift ratios.

Figure 2.16 Definition of yield deformation

Figure 2.17 Definition of ultimate deformation [H1]

2.4.3 Database of beam-column joints with high-strength materials

To evaluate the current code provisions in designing high-strength beam-column joints, all the available test data of specimens constructed with high-strength concrete or high-strength reinforcement are reviewed and investigated in this section. The collected database in this study includes 92 beam-column joint tests that were constructed using HSC and HSS reinforcement. Details and parameters of the considered joints are presented in Appendix I.

The failure modes are categorized as J, BJ and BCJ. A failure mode is designated as J if the joint has failed prior to the yielding of longitudinal beam or column bars, BJ if the longitudinal beam bar has failed prior to joint failure, and BCJ if both beam and column bars yielded before the joint failure. Joints with eccentricity are excluded from the database and all the selected joints consist of concentric beams and columns. **Figure 2.18** shows the range of concrete compressive strength and the reinforcing steel yield strength in beams or columns.

2.4.4 Effect of column axial loading on joint shear stress

The effect of the column axial loading ratio on the joint shear strength has been investigated in literature. However, different studies and experiments have come to contradictory conclusions. Some researches such as Clyde *et al.* [C2], Barnes and Jigoral [B1] and Fujii and Morita [F2] have concluded that increasing the axial loading level will result in an increase in joint shear strength. On the other hand, some studies, such as the research done by Vecchio and Collins [V1], show that the joint shear strength decreases as the axial loading ratio increases. Some other studies, such as those conducted by Pantazopoulou and Bonacci [P1], Vollum [V2], and Kurose *et al.* [K3], show that the joint shear strength is not affected by the axial load ratio. However, most studies show that the ductility capacity is decreased when the specimen is subjected to high axial loading, which results in more rapid strength degradation after failure initiation.

Figure 2.18 Range of the strength of the material for specimens in the database

In the case of beam-column joints constructed with high-strength material, the effect of the axial load ratio can be explored using the collected database. **Figure 2.19** demonstrates the relationship between axial load ratio and joint shear strength coefficient for the specimens used in the database. Considering the general trend, there is a slight increase in the joint shear strength as the axial load ratio is going up to $\frac{P}{f'_c A_g} = 0.15$. The increase in the shear strength of the joints with J-type failure is

clear as the axial load ratio increases. For the joints with BJ-type failure, the trend is less conclusive. The reason is that the beam flexural strength, which is limiting the joint shear at failure, is not affected by axial loads.

It may be observed that the specimens failed in BJ failure mode have achieved higher strength. The reason is that the higher axial load level will increase the joint shear strength, which will lead to the failure initiation at the beam. In addition, higher axial loading increases the bond strength of beam bars in the joint region. **Figure 2.20** illustrates the effect of the axial loading ratio on the displacement ductility capacity. It is observed that in joints with J-failure, increasing the axial loading level will increase the displacement ductility factor clearly. However, as the beam contribution in the failure mechanism is increased in the test specimens with B-J and B failure mode, the influence of the column axial loading level is decayed. This observation could be justified by considering the confining effect of column axial loading on the joint behavior.

2.4.5 Effect of concrete compressive strength on joint shear stress

One of the most important parameters affecting the joint shear behavior is the concrete compressive strength (f'_c), as the joint shear failure happens because of the weakened concrete compressive strength after diagonal tension cracking in the joint panel. It is essential to have a good bond condition in order to make sure that the panel truss mechanism takes place. The high concrete compressive strength (f'_c) is one of the primary factors to make the bond condition achieve its favorable level. However, (f'_c) is the only limiting factor for shear design of beam-column joints in many design codes.

Figure 2.19 The effect of axial load ratio on the joint shear stress coefficient

Figure 2.20 The effect of axial load ratio on the displacement ductility factor

The effect of concrete compressive strength on the joint shear stress for all the database specimens is depicted in **Figure 2.21**. The regression analysis of the data points implies the following relationship between the joint shear stress and the concrete compressive strength:

$$v = 0.15 f'_c (MPa) \quad (2.37)$$

Although NZS 3101-06 [N3] highly relies on the panel truss mechanism and almost ignores the main strut contribution, the joint shear stress limit is set to be $0.2f'_c$ in the code. The limiting value in NZS 3101-06 [N3] and the trend line obtained based on the analysis of the database is shown in **Figure 2.21**.

As observed, a specific relation between the joint shear stress and concrete compressive strength is not clear. As the concrete compressive strength increases, the NZS 3101-06 [N3] limit is becoming more conservative and the joint shear stress has been proven to be much lower than the specified limit in NZS 3101-06 [N3]. The influence of other parameters, such as the joint shear reinforcement, is also significant in the joint shear behavior and further analysis is needed to understand the joint shear strength better.

2.4.6 Effect of beam reinforcement yield strength and reinforcing details on joint shear stress

The influence of the longitudinal beam reinforcement on joint shear stress is crucial as the bars passing through the joint region will help to clamp the opening of the diagonal cracks to a certain extent. In ACI 318-11 [A1], the design of joint shear reinforcement is related to the confined core dimension, the hoops spacing, the yield strength of hoops, and the concrete compressive strength. However, the design of joint reinforcement in NZS 3101-06 [N3] is totally related to the area of the beam bars that pass through the joint region.

The effect of beam reinforcement detailing can be studied according to the collected database. **Figure 2.22** demonstrates the relationship between the joint shear stress factor and the beam bar ratio. A general trend can be observed where the increase in the beam bar ratio will cause a rise in joint shear stress factor. As such, it is common in design codes to specify an upper bound limit for the beam bar ratio to

ensure satisfactory joint shear behavior. Designing a beam-column joint with the presence of excessive beam reinforcement can increase the joint shear stress more than the beam flexural capacity, which will result in a relatively weaker joint rather than the framing members. Therefore, the joint shear failure might occur prior to beam hinging.

Figure 2.23 demonstrated the relationship between the joint shear stress factor and the beam bar yield strength. The general relationship between the beam bar yield strength and joint shear stress factor is not clear, as the joint shear stress obtained is scattered and no direct conclusion can be drawn. In order to study the effect of the beam bar yield strength on the joint shear stress factor, further finite-element parametric study is required, which will be discussed in **Chapter 7**.

In order to better understand the effect of reinforcement area and yield strength of longitudinal beam bars, it is common in literature to consider the beam reinforcement index (BI) as defined below:

$$BI = \frac{A_s f_y}{b.d.f_c'} \quad (2.38)$$

Figure 2.24 depicted the relationship between the joint shear stress coefficient and the beam reinforcement index (BI). The trend is notable where the joint shear strength is rising as the beam reinforcement index is increased. It can be seen that, in the case of a larger reinforcement area or higher yield strength of beam rebars, the joint shear stress will increase and the failure mode may change to joint shear. This conclusion should be further investigated in the experimental program and analytical studies.

Figure 2.21 Joint shear stress versus concrete compressive strength in the database

Figure 2.22 Joint shear stress factor versus beam bar ratio

2.4.7 Effect of joint core reinforcement on joint shear stress

The role of transverse hoops in a beam-column joint is to provide shear strength in addition to confine the core concrete in the joint. The design philosophy of joint shear reinforcement in building codes is to make sure that the joint region has enough shear capacity during earthquake loading. There are quite a number of tested joints in literature that failed in joint shear although they were designed according to code requirements to have the beam hinging failure mode.

In order to study the effect of joint core reinforcement, the joint reinforcement ratio is defined as the total cross-sectional area of the hoops over the total depth in the loading direction divided by the total area obtained from the product of column width and beam depth. **Figure 2.25** illustrates the relationship between the joint shear stress coefficient and the joint core reinforcement ratio according to the existing database. As a general trend, it is observed that the higher joint core reinforcement ratio will decrease the joint shear stress, which will cause the failure mode to change from J-Failure to beam hinging. As observed, most of the specimens with joint shear failure have a joint core reinforcement ratio of less than 0.4%. As the reinforcement ratio goes beyond 0.5%, beam hinging and BJ-failure will become the dominant failure mode.

The general assumption in designing ductile moment frames is to make sure that plastic hinges are formed in beam ends and the columns will remain elastic. Joint hoops are provided to avoid joint failure by providing adequate shear resistance and confinement of the joint region.

Figure 2.23 Joint shear stress factor versus the yield strength of beam bars

Figure 2.24 The effect of beam reinforcement on the joint shear strength coefficient

2.4.8 Effect of bond condition on joint shear stress

The relation between the bond stress along the reinforcement and the bond strength can be studied by defining a bond index factor (μ) that is equal to the normalized maximum beam-bar bond stress in the joint. The bond index can be evaluated according to the following equation:

$$\mu = \frac{\mu_b}{\sqrt{f'_c}} \quad (2.39)$$

where μ_b is the average bond stress and the bond strength is assumed to be equal to the square root of the concrete compressive strength $\sqrt{f'_c}$. The bond index was defined initially based on a single bar passing through the joint. In a case where several bars with different properties and diameters exist in the beam section along the beam-joint surface, the bond index is defined for the rebar that has the largest BI value.

The bond index can be used as a tool to evaluate the failure mechanism. **Figure 2.26** shows the relation of the bond index and the joint shear stress for all of the database specimens.

Li and Leong [L2] recommended that the upper limit of 1.0 for the bond index happens for the lightly reinforced joints while the upper boundary of 1.5 belongs to ductile joints and BI is greater than 2.0 for the joint shear failure. According to the database and the presented data in **Figure 2.26**, joint shear failure has happened in specimens with larger bond index values. However, the bond index ranging from 1.0 to 2.0 almost belongs to the specimens with BJ-failure and ductile joints. As there are no lightly reinforced joints in the database, a bond index smaller than 1.0 is not observed in the figure.

Figure 2.25 Joint shear strength coefficient versus joint core reinforcement ratio

Figure 2.26 Joint shear strength coefficient versus bond index

2.5 Recommendations of Concrete Structures Standards for the Design of Beam-Column Joints

2.5.1 NZS 3101-06 (2006)

The New Zealand Concrete Structures Standard, NZS 3101-06 [N3], provides a design procedure for the interior and exterior joints of fully ductile frames. The area of total effective horizontal joint shear reinforcement in interior joints is presented as follows:

$$A_{jh} = \frac{6V_{jh}}{f'_c b_j h_c} \left(\frac{\alpha_i f_y A_s}{f_{yh}} \right) \quad (2.40)$$

Where $\alpha_i = \left(1.4 - 1.6 \frac{C_j N_0}{f'_c A_g} \right) \alpha_n$; and α_n is the sectional curvature ductility of the

plastic region. C_j is defined as $C_j = \frac{V_{jh}}{V_{jx} + V_{jz}}$.

The required horizontal joint shear reinforcement in exterior joints is expressed as follows:

$$A_{jh} = \frac{6V_{jh}}{f'_c b_j h_c} \left(\frac{\beta f_y A_s}{f_{yh}} \right) \left(0.7 - \frac{C_j N_o^*}{f'_c A_g} \right) \quad (2.41)$$

Where $0.85 \leq \frac{6V_{jh}}{f'_c b_j h_c} \leq 1.20$

The ratio of maximum longitudinal beam bar diameter to column depth is determined by NZS 3101-06 [N3] as follows:

$$\frac{d_b}{h_c} \leq 6 \left(\frac{\alpha_i \alpha_p}{\alpha_s} \right) \alpha_f \frac{\sqrt{f'_c}}{1.25 f_y} \quad (2.42)$$

Where

α_s is a coefficient allowing more severe bond stress conditions at over-strength;

$\alpha_t = 0.85$ for a top bar where more than 300 mm of fresh concrete is cast below the bar;

$\alpha_t = 1.0$ for all other cases;

$\alpha_f = 0.85$ for the beam bars passing through a joint in two directions;

$\alpha_f = 1.0$ for beam bars in one-way frames;

$\alpha_p = \frac{N_0}{2f'_c A_g} + 0.95$; where N_0 is the maximum design over-strength axial load determined by capacity design.

However, the value of f'_c in above equations is limited to 70 MPa and the recommendations are not valid in the case of using HSS reinforcement.

2.5.2 ACI 318-11 (2011)

For the joints of special moment frames, shear strength is not sensitive to the joint shear reinforcement based on ACI 318-11 [A1] recommendations. Joint shear capacity is just related to the concrete compressive strength and the level of joint confinement provided by transverse beams. The upper limit of joint shear strength (MPa) for different cases is defined as below:

$$V_n = 1.66\sqrt{f'_c} b_j h_c; \quad \text{joints confined on all four faces} \quad (2.43a)$$

$$V_n = 1.245\sqrt{f'_c} b_j h_c; \quad \text{joints confined on three faces or on two opposite faces} \quad (2.43b)$$

$$V_n = 0.996\sqrt{f'_c} b_j h_c; \quad \text{other cases} \quad (2.43c)$$

Where f'_c is the concrete compressive strength and the effective area of the joint $A_j = b_j \times h_c$ is as shown in **Figure 2.27**.

Figure 2.27 Definition of the joint effective area in ACI 318-11 [A1]

The total section area of joint rectangular hoop reinforcement shall not be considered less than the values given in **Equation 2.44**.

$$A_{sh} = 0.3 \frac{S \cdot h_c \cdot f'_c}{f_y} \left(\frac{A_g}{A_c} - 1 \right) \quad (2.44a)$$

$$A_{sh} = 0.09 \frac{S \cdot h_c \cdot f'_c}{f_{yh}} \quad (2.44b)$$

Where

A_c is the cross sectional area measured out-to-out of the transverse reinforcement; A_g is the section gross area; f_{yh} is the yield strength of the transverse reinforcement and h_c is the dimension of the column core.

On the other hand, the stress in flexural tensile reinforcement is defined as in **Equation 2.45** in order to determine the forces in longitudinal beam reinforcement at the joint face.

$$T = 1.25 f_y \quad (2.45)$$

The critical issue in the design of beam-column joints with high-strength reinforcing bars is regarding the splice and anchorage length requirements. Utilizing

HSS reinforcement will increase the bond-slip at the joint, which reduces the energy dissipation capacity and the joint shear strength. In order to prevent the excessive bond-slip, ACI 318-11 [A1] has limited the column depth to beam bar diameter in interior joints as follows:

$$\frac{d_b}{h_c} \leq \frac{1}{20} \quad (2.46)$$

In addition, ACI 318-11 [A1] has specified a minimum development length l_{dh} of a 90 degree hook bar in exterior beam-column joints as mentioned in **Equation 2.47**. Under reversed cyclic loading, beam reinforcing bars are subjected to compressive forces as well as tension yield forces. As a result, the straight bar length of the hook bar should satisfy the following equations:

$$l_{dh} \geq f_y d_b / 5.4 \sqrt{f'_c} \text{ (MPa)} \quad (2.47a)$$

$$l_{dh} \geq f_y d_b / 4 \sqrt{f'_c} \text{ (MPa)} \quad (2.47b)$$

Where **Equation 2.47a** is the required tension development length and **Equation 2.47b** is the required compressive development length.

The current ACI 318-11 [A1] allows the use of reinforcement with the yield strength of 550 MPa for flexure and confinement. However, the specified strength of reinforcement is limited to 420 MPa in special moment frames. The objective of this study is to identify provisions of ACI 318-11 [A1] that need to be modified to allow the use of HSS reinforcing bars for moment frames and members.

2.5.3 Eurocode 2 (2004)

In Eurocode 2 [E4], the calculation of the required anchorage length shall take into consideration the type of steel and bond properties of the bars. The design anchorage length is calculated using the following equation:

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \quad (2.48)$$

Where α values are the coefficients defined in Eurocode 2 [E4] and $l_{b,rqd}$ is the basic required anchorage length.

In case of using large diameter bars in design of RC members, additional rules for bars with a diameter larger than 32 mm have been defined in Eurocode 2 [E4]. When such large diameter bars are used, following rules need to be satisfied:

1. Crack control should be achieved by using surface reinforcement or by calculation as described in the code.
2. Splitting forces are higher and dowel action is greater with the use of large diameter bars. Such bars should be anchored with mechanical devices. As an alternative they may be anchored as straight bars, but links should be provided as confining reinforcement.
3. Transverse reinforcement, additional to that for shear, should be provided in the anchorage zones where transverse compression is not present. The additional reinforcement should not be less than the following:

$$A_{sh} = 0.25A_s n_1, \text{ in the direction parallel to the tension face} \quad (2.49a)$$

$$A_{sv} = 0.25A_s n_2, \text{ in the direction perpendicular to the tension face} \quad (2.49b)$$

Where A_s is the cross sectional area of an anchored bar, n_1 is the number of layers with bars anchored at the same point in the member, and n_2 is the number of bars anchored in each layer.

4. The additional transverse reinforcement should be uniformly distributed in the anchorage zone and the spacing of bars should not exceed 5 times the diameter of the longitudinal reinforcement.

2.6 Recent Experimental Investigations on Beams and Columns with HSS Reinforcement

A number of cycling tests has been conducted on beams constructed with high-strength reinforcement. In 1980s and 1990s, Japanese researchers studied the use of ductile reinforcing bars with specified yield strength of greater than 550 MPa. A summary of these efforts, which includes more than 25 laboratory reports and 105 beam test specimens, has been presented by Otani *et al.* [O2]. In addition, Aoyama

[A6] presented the summary and major findings of the New Rc Project. Sugano *et al.* [S6] tested two beams reinforced with HSS longitudinal and transverse steel having yield strength of 586 and 862 MPa, respectively. In a recent study by Tavallali [T1], cyclic testing of concrete beams reinforced with 675 MPa rebars was conducted. As a summary of all the conducted tests on beams with HSS reinforcements, it was concluded that replacing Grade 420 MPa longitudinal reinforcement with Grade 700 MPa reinforcement leads to comparable flexural strength and deformation capacity. Researchers suggested that concrete beams reinforced with HSS reinforcement could be a practical option in earthquake-resistant design of RC frames. This advantage could be achieved if brittle failures due to bond, bar buckling, and joint shear distress are avoided and flexural yielding is dominated.

In recent years, cyclic tests have been conducted on concrete columns reinforced with high-strength steel. Restrepo *et al.* [R5] tested two circular columns with Grade 420 MPa and 700 MPa reinforcement. The columns were made with self-consolidating concrete with a concrete compressive strength of 55 MPa. Test results indicated that columns with HSS reinforcement show an acceptable behavior until a drift level of 3.9%. Rautenberg [R2] tested concrete columns with Grade 420 to 850 MPa longitudinal reinforcement. It was concluded that utilizing HSS longitudinal bars in the column section leads to comparable flexural strength and deformation capacity.

Basically, many tests have been conducted in Japan on columns with yield strength of the longitudinal and transverse reinforcement reaching 1200 MPa and compressive strength of concrete ranging from 55 to 200 MPa. Ishikawa *et al.* [I2] calibrated statistical models for the ultimate drift capacity of columns using a large database of 115 columns.

2.7 Research on High-Strength Reinforcement in Japan

In 1990s, the New RC Project was conducted in Japan to study high-strength reinforcement as described by Aoyama [A6]. Specifications of high-strength reinforcement were introduced in Japan in 1995 which coincided with the rapid increase in the number of constructed reinforced concrete high-rise buildings.

Sugano *et al.* [S5] described that approximately 200 high-rise buildings were constructed in Japan between 1995 to 2002. Based on these studies, four different types of high-strength reinforcing bars are developed in Japan as follows:

1. USD685A and USD685B with yield strength of 690 MPa. These reinforcing bars are expected to yield in beams and columns;
2. USD980 with yield strength of 980 MPa which are used in beams and columns not expected to yield;
3. USD785 with yield strength of 786 MPa;
4. USD1275 with yield strength of 1275 MPa which is intended for use as transverse reinforcement.

These new reinforcement types have not yet been accepted into the Japanese Industrial Standard (JIS). However, these reinforcement types have gained acceptance through the Ministry of Construction as part of the New RC Construction Standard as presented by Aoyama [A6].

Common methods for producing high-strength reinforcement include cold working, addition of alloys, and quenching and tempering. In terms of the material properties, tensile strength and elongation are among the important measures of the suitability of the reinforcing bars for use in members resisting earthquake effects. Japan's New RC Project explored the effect of the tensile strength to yield strength ratio in a series of beam tests. The tests compared the shapes of hysteresis loops for cyclically loaded beams constructed using reinforcement with various levels of this strength ratio. As demonstrated by the New RC Project tests, tensile characteristics and the shape of stress-strain curves affect the cyclic behavior of members.

2.8 Conclusions

In this chapter, additional research needed to support a recommendation for the use of HSS reinforcement was identified. According to the previous experimental studies on the behavior of reinforced concrete beam-column joints constructed using high-strength materials, a database that includes test data on 92 specimens has been collected and presented in Appendix I. A parametric study on the collected database was presented in this chapter. The following conclusions were made based on the literature review:

1. The existing research was reviewed in this chapter to determine if the experimental and analytical information are sufficient to justify the use of reinforcement with the minimum yield strength of 550 MPa for special moment frames.
2. Development lengths have been investigated in literature for steel bars with yield strength of up to 1034 MPa, but the concrete compressive strength in these tests was below 70 MPa. Additional splice tests are required to determine the development lengths for HSS reinforcing bars placed in concrete with strength of up to 100 MPa.
3. Design rules regarding the use of HSS reinforcing bars in moment frames need to develop. Additional testing of interior and exterior joints is required to determine the joint depths for reinforcement with yield strength of 550, 700 and 830 MPa. The required joint depth should be related to the reinforcement yield strength and the concrete compressive strength.
4. Tests on HSC beam-column joints revealed that the concrete compressive strength was the major parameter that controlled the behavior of joints as less cracks appeared in HSC specimens. The modes of failure and shear behavior of these joints behaved differently from normal-strength concrete joints at various axial load levels. It was concluded that the stiffness degradation of the HSC specimens was less severe than that in normal-strength concrete specimens.
5. Previous parametric studies carried out on the experimental database demonstrated that the joint shear strength increases as the longitudinal beam reinforcement ratio, the yield strength of longitudinal reinforcement, and the concrete cylinder strength increases.
6. Bond tests revealed that the bond strength in HSC joints is higher than that of corresponding normal-strength specimens. However, the bond behavior is more brittle in HSC specimens. Further research could be conducted to study the bond behavior of HSS reinforcement in HSC beam-column joints.
7. The results of cyclic testing indicated that an increase in lateral displacement

- will lead to more severe bond damage. Cyclic loading does not affect the bond strength of HSC as long as the cyclic slip is less than the maximum slip for monotonic loading. The behavior of HSC under cyclic loading is slightly different from that of normal-strength concrete.
8. Most experiments assumed no axial load existed in the test, which was albeit the most critical case that led to conservatism in the design. However, the presence of axial load is inevitable and the practical way to conduct a test is to have the minimum axial load to represent the actual condition.

CHAPTER 3

EXPERIMENTAL PREPARATION AND TEST PROCEDURE FOR BEAM-COLUMN JOINTS

3.1 Introduction

As concluded in the literature review chapter, it is essential to provide more experimental data in order to investigate the seismic behavior of beam-column joints constructed using high-strength concrete (HSC) coupled with high-strength steel (HSS) reinforcement. The effect of the column axial loading and also the investigation on bond performance of HSS bars are to be conducted in this experimental program. A total number of 12 interior and exterior beam-column joints were designed, constructed and tested in the Protective Engineering (PE) Laboratory at Nanyang Technological University (NTU). An explanation to the experimental program is presented in this chapter.

3.2 Description of Test Specimens

3.2.1 Details of test specimens

Seven interior and five exterior specimens of the same dimensions but different reinforcement details and different strength of the materials were cast in place to present a typical beam-column joint of an RC building structure. The interior specimens were labeled as IN80, IH80, IH80A, IN100, IH100, IH60, and IH60A. The exterior specimens were labeled as EN80, EH80, EH80A, EH60 and EH60A. The letter “H” stands for high-strength steel reinforcement in specimens and letter “A” indicates the presence of column axial compression loading. The grade of concrete in each specimen is shown as a number in its label. The beam span was 4.8m and the height of the column was 3.3m in all of the testing specimens. The specimens were designed based on the strong column-weak beam requirement and detailing requirements of ACI 318-11 [A1] and ACI 352-02 [A3]. The seismic design method is based on limiting the joint shear stress values to have a beam flexural failure mechanism. Joint shear failure is not expected in the designed

specimens. These seven specimens were identical to the specimens with concrete grade of 60 MPa and reinforcement grade of 500 MPa tested by Li and Leong [L2].

3.2.2 Reinforcement detailing

As shown in **Figures 3.1** through **3.5** for interior specimens, beams were reinforced using 2T16 (16 mm diameter) bars at the bottom and 4T16 at the top in Specimens IN80, IH80, IH80A, IN100 and IH100. However, in Specimens IH60 and IH60A, 2T19 bars were used as the top and bottom beam reinforcements. Beam bars were continuous through the entire span. In columns of Specimens IN80 and IN100, a combination of T25 and T16 bars were used as longitudinal reinforcement, while a combination of T25, T24 and T16 bars were utilized in all the other specimens. Transverse reinforcements in beams and columns were designed according to the provisions of ACI 318-11 [A1] for the purpose of resisting shear, confining concrete and preventing premature longitudinal bar buckling. Transverse reinforcements in the joint region were designed according to the recommendations of ACI 352-02 [A3]. Hence, 5 sets of normal-strength hoops with a spacing of 65 mm were used in the joint of Specimens IN80 and IN100 while 4 sets of HSS hoops with a spacing of 85 mm were utilized in Specimens IH80, IH80A, IH60 and IH60A. However, 5 sets of HSS hoops with a spacing of 65 mm were used in Specimen IH100 with the concrete grade of 100 MPa. **Table 3.1** summarizes the design parameters of interior specimens. As shown in **Figures 3.6** through **3.8** for exterior specimens, beams were reinforced using 2T16 (16 mm diameter) bars at the bottom and 4T16 at the top in Specimens EN80, EH80 and EH80A. However, in Specimens EH60 and EH60A, 2T19 bars were used as the top and bottom beam reinforcements. Beam bars were anchored inside the joint region. In Specimen EN80, a combination of Grade 500 bars of diameter T20 and T16 was used in the column section as longitudinal reinforcements, while a combination of Grade 700 bars of diameter T22, and Grade 500 bars of T16 were utilized in the column design of all the other specimens. Similar to interior specimens, 5 sets of HSS hoops with a spacing of 65 mm were used in the joint of Specimens EN80, EH80 and EH80A while 4 sets of the same hoops with a spacing of 85 mm were utilized in Specimens EH60 and EH60A. **Table 3.2** summarizes the design parameters of the exterior specimens.

Table 3.1 Summary of interior test specimens

Property		IN80	IH80	IH80A	IN100	IH100	IH60	IH60A
Concrete f'_c (MPa)		80	80	80	100	100	60	60
Beam	Dimensions	$250 \times 500mm$						
	Largest bar diameter (mm)	16	16	16	16	16	19	19
	Reinforcement grade	500	700	700	500	700	700	700
	Top bar area ρ (%)	0.71	0.71	0.71	0.71	0.71	0.5	0.5
	Bottom bar area ρ' (%)	0.36	0.36	0.36	0.36	0.36	0.5	0.5
	$\beta = A'_s / A_s$	0.5	0.5	0.5	0.5	0.5	1	1
Column	Dimensions	$300 \times 450mm$						
	Largest bar diameter (mm)	25	25	25	25	25	25	25
	Reinforcing bar ratio (%)	2.05	2.59	2.59	2.05	2.59	2.59	2.59
	Corner bars grade	500	700	700	500	700	700	700
	Middle bars grade	500	500	500	500	500	500	500
	Axial load ratio	0	0	0.3	0	0	0	0.3
Joint	Hoops diameter (mm)	10	10	10	10	10	10	10
	Hoops grade	500	700	700	700	700	700	700

Table 3.2 Summary of exterior test specimens

Property		EN80	EH80	EH80A	EH60	EH60A
Concrete f'_c (MPa)		80	80	80	60	60
Beam	Dimensions	$250 \times 500mm$				
	Largest bar diameter (mm)	16	16	16	19	19
	Reinforcement grade	500	700	700	700	700
	Top bar area ρ (%)	0.71	0.71	0.71	0.5	0.5
	Bottom bar area ρ' (%)	0.36	0.36	0.36	0.5	0.5
	$\beta = A'_s / A_s$	0.5	0.5	0.5	1	1
Column	Dimensions	$300 \times 450mm$				
	Largest bar diameter (mm)	20	22	22	22	22
	Reinforcing bar ratio ρ_c (%)	2.05	2.59	2.59	2.59	2.59
	Corner bars grade	500	700	700	700	700
	Middle bars grade	500	500	500	500	500
	Axial load ratio $(P / f'_c A_g)$	0	0	0.3	0	0.3
Joint	Hoops diameter (mm)	10	10	10	10	10
	Hoops grade	700	700	700	700	700

*All dimensions are in *mm*

Figure 3.1 Dimensions and reinforcement details of Specimen IN80

*All dimensions are in *mm*

Figure 3.2 Dimensions and reinforcement details of Specimens IH80 and IH80A

*All dimensions are in *mm*

Figure 3.3 Dimensions and reinforcement details of Specimen IN100

*All dimensions are in *mm*

Figure 3.4 Dimensions and reinforcement details of Specimen IH100

*All dimensions are in *mm*

Figure 3.5 Dimensions and reinforcement details of Specimens IH60 and IH60A

*All dimensions are in *mm*

Figure 3.6 Dimensions and reinforcement details of Specimen EN80

*All dimensions are in *mm*

Figure 3.7 Dimensions and reinforcement details of Specimens EH80 and EH80A

*All dimensions are in *mm*

Figure 3.8 Dimensions and reinforcement details of Specimens EH60 and EH60A

3.2.3 Material properties

The target concrete compressive strength for the specimens was 60 and 80 MPa for exterior joints and 60, 80 and 100 MPa for interior joints. The tested values of concrete compressive strength was 60.8, 61.7, and 58.8 MPa for Grade 60 concrete, 81.3, 81.7, and 79.1 MPa for Grade 80 concrete, and 100.2, 101.9, and 98.7 MPa for Grade 100 concrete. Deformed steel bars of yield strength $f_y = 550$ MPa and $f_y = 700$ MPa were utilized as longitudinal reinforcement in beams and columns. The ratio of ultimate strength to yield strength of Grade 500 bars was $f_u/f_y = 1.17$ for T16, 1.25 for T20 and 1.28 for T25. The f_u/f_y ratio for Grade 700 bars was 1.3 for T16, 1.27 for T19, 1.28 for T22 and 1.29 for T25. The transverse reinforcement utilized in beams and columns included R6 and R10 mild steel bars with $f_y = 500$ MPa and $f_y = 700$ MPa. The reinforcing bars were tested under uniaxial tension and the measured parameters of the steel bars are mentioned in **Table 3.3**. The reported values of yield strength and ultimate strength of reinforcing bars are the average values of three tested bars.

Table 3.3 Material properties of reinforcing steel

Grade of steel	500					700				
Bar size	R6	R10	T16	T20	T25	T10	T16	T19	T22	T25
Yield strength, f_y (MPa)	364	563	564	552	506	722	712	708	711	719
Yield strain, $\mu\epsilon$	2157	2254	3700	2911	3984	3760	4618	4084	4006	4314
Ultimate strength, f_u (MPa)	516	650	662	691	651	885	926	902	914	929

The specification of the Japanese steel needs to be summarized in this section carefully. The percentage of carbon is 0.24% and 0.32% in Grade 500 and 700 MPa steel, respectively. The percentage of phosphorus, sulfur, nitrogen, and copper is 0.055, 0.055, 0.014, and 0.85 in steel reinforcement with both grades. The behavior of test specimens, especially in terms of the bond performance, is quite sensitive to the surface properties of reinforcing bars. These parameters include the type of deformation, depth of rib, and spacing of the rib. Surface properties of the reinforcing bars are summarized in **Table 3.4**.

Table 3.4 Surface properties of reinforcing steel

Bar size (mm)	Nominal dimension		Outside diameter	Inside diameter		Dimension of transverse rib	
	Diameter (mm)	Cross-section area (mm ²)		B(mm)	C(mm)	P(mm)	H(mm)
16	16	201	17.1	14.9	14.1	7	1.5
19	19	314	22.4	18.3	18.8	8	1.8
22	22	380	24.5	19.5	20.3	9	2.1
25	25	491	28.1	22.9	23.5	10	2.3

The reinforcement was hot rolled threaded rebars and the threaded geometry was on all over the rebars surface. The advantage of using these threaded rebars is that strain gauges could be attached to their surface conveniently due to a space between the threads. In the current experimental program, numerous strain gauges were attached along the reinforcing bars without losing the cross-section area as the surface of rebars is not grinded excessively. In addition, the other advantage of these rebars is that they can be cut at any point along its length and joined easily with a coupler using the principle of screw.

3.2.4 Fabrication of test specimens

The specimens were cast in five batches as the specimens with the same geometry and concrete grade were cast in the same batch. Steel cages were prepared with strain gauge installed for the measurement of strain development during the test. All the formwork and the moulds were moistened before casting. During casting, compaction was achieved by using a 1-inch poker vibrator to ensure good flow of concrete around the reinforcement. **Figures 3.9** and **3.10** show the steel cage of specimens as well as the casting process. The specimens were demoulded after three days and were left for air curing for at least three weeks before testing.

In order to obtain the properties of concrete, cylinders of ϕ 150 mm x 300 mm (height) were prepared. Strength development of concrete was observed on the 7th and 28th day based on the average strength of three cylinders.

3.3 Test Setup and Loading Procedure

The test setup is illustrated in **Figures 3.11** and **3.12** for interior and exterior test specimens, respectively. The bottom of the column is pinned to the strong floor and beam ends are connected to the strong floor using two vertical steel links, which only restrain the beams vertical movements and permits the rotation and free horizontal translation of beams. Each test specimen was subjected to quasi-static reversed cyclic simulated earthquake loading as shown in **Figure 3.13**. The horizontal cyclic loading applied to the specimens throughout the test was displacement-controlled. Test specimens were subjected to one cycle of loading with displacement equivalent to story drift ratio (DR) of 0.5, 0.75 and 1.0% followed by two cycles to each of 1.5%, 2.0%, 2.5%, 3.0%, 3.5% and 4.0%. The story drift ratio is defined as the inter-story horizontal displacement divided by the story height. The horizontal loading is applied to the top of the column using a 1000 kN capacity hydraulic actuator. Constant compression axial loading of magnitude $0.3 f'_c A_g$ was applied on the column of Specimens IH80A, IH60A, EH80A and EH60A, while no axial load was applied for other specimens.

3.4 Instrumentation

All the test specimens were equipped with numbers of strain gauges and linear variable displacement transducers (LVDTs) in order to measure strain values in selected points on reinforcing steel bars as well as the deformation of different parts of the specimens. As depicted in **Figures 3.14** and **3.15**, twenty LVDTs within interior beam-column joints and ten LVDTs within exterior beam-column joints were installed to measure the flexural deformation of beams and columns. In addition, two diagonal LVDTs were used in all the joint panels in order to measure the joint shear deformation. Two more LVDTs were placed on top of the column and the beam end to record the horizontal displacements.

Figure 3.9 Construction of interior specimens

Figure 3.10 Construction of exterior specimens

Figure 3.11 Loading rig for interior specimens

Figure 3.12 Loading rig for exterior specimens

Figure 3.13 Displacement history applied to the test specimens

Strain gauges were installed at selected positions on both longitudinal and transverse reinforcements within and around the joint region. All the instruments were connected to the data acquisition system to record the data during the test. As a result of such measurements, hysteresis loop of specimens which shows the relationship between the story shear and horizontal displacements, and also the strain distribution and deformation values can be reported. The mentioned findings are used to analyze the seismic behavior of HSC beam-column joints reinforced with HSS.

3.4.1 Measurement of horizontal displacement

The displacement of different parts of the specimens could be measured in order to find the external deformations. According to the displacement applied to the specimens, two types of transducers of 300 mm and 100 mm travel distance are used to measure the displacement of different parts. A displacement transducer with 300 mm travel is used at the top column end-pin to measure the horizontal displacement. In addition, a displacement transducer with 300 mm travel is used at the end of the beam in order to measure the rigid body movement. To measure vertical displacements, two transducers with the displacement capacity of 100 mm are installed on the bottom of the beams. These transducers are put in the points where the maximum vertical displacement is expected. Finally, a transducer with the displacement capacity of 100 mm is installed on the level of the bottom end of the column to measure the lateral displacement.

Figure 3.14 Positions of linear potentiometers and strain gauges in interior joints

* All dimensions are in *mm*

Figure 3.15 Positions of linear potentiometers and strain gauges in exterior joints

3.4.2 Measurement of rotation of beams and columns

50 mm displacement transducers were used to measure the rotation of beams and columns. There are two types of beam or column rotations which can be measured using transducers; the fixed end rotation and the flexural rotation at a particular region. In general, the fixed end rotation and the flexural rotation are obtained by measuring the difference between the displacement of top and bottom transducers in respective to the vertical distance between the transducers.

As a result, the beam fixed end rotation is derived as below:

$$\theta_{b,fe} = \frac{{}_t\delta_1 - {}_b\delta_1}{h_1} \quad (3.1)$$

where:

${}_t\delta_1$ is the top displacement measured over the 1st region in the beam;

${}_b\delta_1$ is the bottom displacement measured over the 1st region in the beam;

h_1 is the distance between top and bottom transducers in the 1st region in the beam.

Figure 3.16 Estimation of fixed-end rotation, curvature and flexural deformation

The beam flexural rotation at a particular region $\theta_{b,i}$ is estimated as below:

$$\theta_{b,i} = \frac{{}_t\delta_i - {}_b\delta_i}{h_i} \quad (3.2)$$

where:

${}_t\delta_i$ is the top displacement measured over the region i in the beam as shown in **Figure 3.16**;

${}_b\delta_i$ is the bottom displacement measured over the region i in the beam;

h_i is the distance between top and bottom transducers in the region i in the beam.

Similarly, in order to obtain the column fixed end rotation ($\theta_{c,fe}$) and the flexural rotation ($\theta_{c,i}$), the difference in displacement of left and right transducers could be divided by the horizontal distance of the transducers.

$$\theta_{c,fe} = \frac{{}_r\delta_1 - {}_l\delta_1}{h'_1} \quad (3.3)$$

where:

${}_r\delta_1$ is the right displacement measured over the 1st region in the column;

${}_l\delta_1$ is the left displacement measured over the 1st region in the column;

h'_1 is the distance between left and right transducers in the 1st region in the column.

$$\theta_{c,i} = \frac{{}_r\delta_i - {}_l\delta_i}{h'_i} \quad (3.4)$$

where:

${}_r\delta_i$ is the right displacement measured over the region i in the column;

${}_l\delta_i$ is the left displacement measured over the region i in the column;

h_i is the distance between left and right transducers in the region i in the column.

3.4.3 Measurement of curvature of beams and columns

The estimation of curvatures in beams and columns is obtained as below:

$$\phi_{b,i} = \frac{\theta_{b,i}}{s_i} \quad (3.5)$$

where:

$\theta_{b,i}$ is the rotation measured over the region i in the beam;

s_i is the gauge length of the region i in the beam.

$$\phi_{c,i} = \frac{\theta_{c,i}}{s_i} \quad (3.6)$$

where:

$\theta_{c,i}$ is the rotation measured over the region i in the column;

s_i is the gauge length of the region i in the column.

3.5 Components of Horizontal Displacement

The horizontal displacement of the test specimens consists of the deformation contributions from beams, columns and joints. The procedure to estimate the contribution of each component is described in following sections.

3.5.1 Beam deformations

The deformations of the beams are composed of the flexural deformation ($\delta_{b,f}$) as well as the beam fixed end deformation ($\delta_{b,fe}$).

Flexural deformations

Flexural deformations of the beams ($\delta_{b,f}$) were obtained from the flexural rotation of each region in the beam ($\theta_{b,i}$). Hence, the beam flexural deformation can be presented as follows:

$$\delta_{b,f} = \sum (\theta_{b,i}) (l'_b - x_i) \quad (3.7)$$

where:

$\theta_{b,i}$ is the beam flexural rotation of the region i ;

l'_b is the distance from the column face to the center of the beam end pin;

x_i is the distance from the column face to the center.

The horizontal displacement at the column top due to the beam flexural displacement ($\Delta_{b,f}$) is estimated as follows:

$$\Delta_{b,f} = \delta_{b,f} \frac{h}{l} \quad (3.8)$$

where:

$\delta_{b,f}$ is the flexural deformations of the beams;

h is the story height or vertical distance between the column end pins (3300 mm);

l is the beam span or horizontal distance between the beam end pins (4800 mm).

Fixed-end rotation

The component of the horizontal displacement due to the beam fixed-end rotation is presented in the following expression:

$$\delta_{b,fe} = \theta_{b,fe} l'_b \quad (3.9)$$

where:

$\theta_{b,fe}$ is the beam fixed-end rotation;

l'_b is the distance from the column face to the center of the beam end pin.

The horizontal displacement at the column top due to beam fixed-end rotation ($\Delta_{b,fe}$) can be calculated based on fixed-end deformations of beams ($\delta_{b,fe}$):

$$\Delta_{b,fe} = \delta_{b,fe} \frac{h}{l} \quad (3.10)$$

where:

h is the story height or vertical distance between the column end pins (3300 mm);

l is the beam span or horizontal distance between the beam end pins (4800 mm).

3.5.2 Column deformations

The deformations of the columns are also composed of the column flexural deformation ($\delta_{b,f}$) as well as the column fixed end deformation ($\delta_{b,fe}$).

Flexural deformations

The column flexural deformation component of the horizontal displacement can be estimated as follows:

$$\Delta_{c,f} = \delta_{c,f} = \sum (\theta_{c,i}) (l'_c - y_i) \quad (3.11)$$

where:

$\theta_{c,i}$ is the column flexural rotation;

l'_c is the distance from the beam face to the column end pin;

y_i is the distance from the beam faces to the center of the region i .

Fixed-end rotation

The component of horizontal displacement due to fixed-end rotation of the column is measured as follows:

$$\Delta_{c,fe} = \delta_{c,fe} = \theta_{c,fe} l'_c \quad (3.12)$$

where:

$\theta_{c,fe}$ is the column fixed-end rotation;

l'_c is the distance from the beam face to the center of the column end pin.

3.5.3 Joint deformations

The joint shear distortion contributes towards the horizontal displacement of the specimens. **Figure 3.17** illustrates the shear deformation of the joints when the beam and column ends are not supported. In the current study, the beams and columns are supported as explained in this chapter. Hence, the horizontal displacement of the specimens due to the joint shear distortion can be derived as follows:

$$\Delta_j = \gamma_j \left(h - h_b - \frac{h}{l} h_c \right) \quad (3.13)$$

where:

γ_j is the joint shear distortion;

h is the story height or vertical distance between the column end (3300 mm);

l is the beam span or horizontal distance between the beam end pins (4800 mm);

h_b is the depth of the beam;

h_c is the column depth.

Figure 3.17 Measurement of joint shear distortion and expansion

3.6 Conclusions

1. This chapter described the design, construction and testing method of the beam-column joint specimens. Seven interior and five exterior joints constructed using high-strength material were tested under reversed cyclic loading.
2. In total, eight specimens were tested with zero column axial loading. However, four specimens were designed to test with the presence of column axial compression loading of $0.3 f'_c A_g$.
3. Specimens IN80, IN100, IH60, and IH60A met the development length requirement of ACI 318-11 [A1] and NZS 3101-06 [N3], while Specimens IH80, IH80A, and IH100 only satisfied the ACI 318-11 [A1] requirements.
4. The test set-up and loading arrangement was introduced. In addition, the method and instrumentation to measure the applied forces, displacements and strains were described.

CHAPTER 4

EXPERIMENTAL RESULTS FOR INTERIOR JOINTS

4.1 Introduction

This chapter presents the test results for each interior joint specimen so as to reflect the behavior of specimens under reversed cyclic loading. The interior specimens have been divided into three groups according to their reinforcement detailing as follows:

- Group one: Specimens IN80, IH80 and IH80A
- Group two: Specimens IN100 and IH100
- Group three: Specimens IH60 and IH60A

The test results were obtained from the readings measured by strain gauges and instruments as explained in **Chapter 3**. The general behavior of the specimens is presented based on the observed cracking pattern, the measured load-displacement hysteresis response and yielding profiles and curvature values in beams and columns. The experimental results for exterior joints are explained in **Chapter 5** and the further discussion of test results is presented in **Chapter 6**.

4.2 Failure Mode of Test Specimens

The failure mechanism of test specimens is described in this section based on the observed cracking pattern, buckling of compressive reinforcement, fracture of tensile reinforcement, and weakening of confined core. The failure mode of all the specimens is beam flexural (BF) failure. In Specimens IH80A and EH80A, fracture of bottom beam reinforcement is observed at final loading cycles. In all the other test specimens, buckling of bottom beam bars was observed at the end of the test. Concrete spalling in the beam plastic hinge region is observed after the buckling or fracture of beam reinforcing bars in all the specimens.

4.3 Test Result of Group One (IN80, IH80 and IH80A)

Group one consists of specimens IN80, IH80 and IH80A with the same dimensions as described in **Chapter 3**. Specimen IN80 was an HSC beam-column joint with

normal-strength steel reinforcement tested without column axial compression loading while IH80 was reinforced with high-strength steel (HSS). Specimen IH80A had the same design and configuration as IH80 but was tested with column axial compression loading of $0.3 f'_c A_g$.

The typical features of these specimens are summarized as follows:

- Smallest diameter of top and bottom bars in the beam section = 16 mm
- Actual maximum beam-bar to column-depth ratio = 1/28
- Actual column-bar to beam-depth ratio = 1/25
- Calculated maximum beam-bar to column-depth ratio based on Eq. 9-21 in NZS 3101-06 [N3] :

$$\text{IN80} = 1/21$$

$$\text{IH80, IH80A} = 1/30$$

- Calculated maximum column-bar to beam-depth ratio based on clause 10.4.6.6 in NZS 3101-06 [N3]:

$$\text{IN80} = 1/17$$

$$\text{IH80, IH80A} = 1/24$$

- Specimen IN80 met the both requirements of maximum beam-bar to column-depth and maximum column-bar to beam-depth ratios set in NZS 3101-06 [N3], whereas IH80 and IH80A did not satisfy the requirement of maximum beam-bar to column-depth ratio.
- Joint core reinforcement detailing complied with the requirement in clause 15.4.4.4 in NZS 3101-06 [N3] (maximum bar spacing = 65 mm, allowable bar spacing = 160 mm)

The following sections report the test result of Specimens IN80, IH80 and IH80A.

4.3.1 Test Results of Specimen IN80

4.3.1.1 General behavior

In general, Specimen IN80 showed a stable energy dissipation capacity. The final cracking pattern of the specimen and the story shear force versus horizontal displacement relationship are shown in **Figures 4.1** and **4.2**, respectively. In addition, the theoretical maximum story shear force P_y calculated based on the development of beam plastic hinges has been illustrated in **Figure 4.2**. The specimen reached its theoretical capacity at a DR of 1.5% in the both negative and positive loading directions. Maximum capacities of 105 kN in the positive and 110

kN in the negative loading directions were reached at a DR of 2.0% and 3.0%, respectively. At DR=4.0%, the specimen experienced a substantial loss of more than 20% of its load-carrying capacity. Up to the final loading cycle, the residual load-carrying capacity in the positive and negative loading direction was 59 kN and 62 kN respectively, which is approximately 56% and 60% of the theoretical capacities.

The crack development of IN80 is illustrated in **Figure 4.3** by a series of photographs taken at the peak of story drift ratios of 1.0%, 2.0%, 3.0% and 4.0%. Flexural cracks appeared corresponding to a DR of 0.5%. In the subsequent loading cycles, the flexural cracks in the positive and negative loading directions propagated in a vertically upward or downward direction covering full depth of the beam and spreading along the beam span. In the load cycle to DR=1.0%, some cracks changed direction and became diagonally oriented when they penetrated into the central part of the beam web. It is noted that the region in which the vertical flexural cracks became diagonal is within the spacing between the distributed longitudinal reinforcements. Some diagonal cracks were distributed in the joint panel and the flexural cracks were observed on column faces. By DR=2.0%, the crack pattern was denser in the joint region as well as the beams. In addition, the cracks in plastic hinge region of beams widened. Additional shear-flexural cracks occurred in the beam web close to beam supports. More new diagonal cracks observed in joint region and corner to corner diagonal cracks occurred in both loading directions. Few cracks developed in top and bottom columns away from the joint region. As the test proceeded toward DR=3.0%, flexural cracks in beam plastic hinge region continue to widen. The cover concrete at the bottom beam chord at column faces showed incipient crushing. The joint panel remained in the crack pattern without too much change. In the load cycle to DR=4.0%, damage on top and bottom beam chords adjacent to column faces spread further along beam span away from column faces and also toward the center of the beam height so that some parts of longitudinal beam bars were exposed. This observation is an indication that bond failure had occurred along these beam bars.

Figure 4.1 Final crack pattern of Specimen IN80

Figure 4.2 Story shear force versus horizontal displacement for Specimen IN80

(a) Drift Ratio of 1.0%

(b) Drift Ratio of 2.0%

(a) Drift Ratio of 3.0%

(b) Drift Ratio of 4.0%

Figure 4.3 Progressive cracking development of Specimen IN80

Figure 4.4 Energy dissipation of Specimen IN80

Furthermore, the energy dissipation of the specimen is illustrated in **Figure 4.4**. The data is shown for the first and second cycles of the loading pattern. It is observed that the values are decreasing after the DR of 3.5% and 3.0% for the first and second loading cycles, respectively. The energy dissipation value at DR=4.0% in the first loading cycle is 51% higher than that in the second cycle.

4.3.1.2 Beam behavior

The evaluation of strains in beam reinforcing bars and the beam curvature values are discussed in the sections that follow. Strain profiles are shown for the positive and negative loading directions to DRs of 1.0% and 3.0% to represent the early loading and late loading stage.

Longitudinal beam bar strains

The strain profiles of the top and bottom beam reinforcement of Specimen IN80 are explained in **Figure 4.5**. As illustrated, the beam bar of IN80 started to yield in tension at the column face after the DR of 3.0%. This is compatible with the observed plastic hinge in beam ends. However, the strain gauges at the center of the joint showed strain values smaller than the yielding strain value throughout the test. This observation clarifies the presence of a good bond in the joint region because of the bar size used in both top and bottom reinforcement.

Beam curvature distributions

Figure 4.6 shows the beam curvature distribution estimated from the transducer readings. Curvature values are presented for drift ratios of DR=1.0% and DR=3.0% in the positive and negative loading directions and yield curvatures are presented as dashed lines. It is observed that beam curvatures measured near the column face reached the theoretical yield value after a DR of 1.0%. However, the rapid increase of beam curvatures was observed in the subsequent loading cycles mainly because of the plastic hinges forming in beam ends.

4.3.1.3 Column behavior

The evaluation of strains in the column side and middle reinforcing bars as well as the column curvature values are discussed in the following sections. Strain profiles are shown for the positive and negative loading directions to DR=1.0% and DR=3.0% to represent the early and late loading stages.

* All dimensions are in *mm*

Figure 4.5 Strain profiles of beam bars in Specimen IN80

* All dimensions are in *mm*

Figure 4.6 Curvature distribution of beams of Specimen IN80

Longitudinal column bar strains

Figure 4.7 demonstrates the strain distribution along the side and central column bars. It is observed that during the positive loading cycles, the side reinforcing bars experience tension in the lower column and compression in the upper side.

In the negative loading direction, side bars experience tension in the upper column and compression in the lower side. In higher drift ratios, the strain values are slightly higher. However, strain values are much smaller than the yielding strain. On the other hand, the central column bars experienced lower stress compared to side bars. In addition, center bars remained under compression during the test.

Column curvature distributions

Figure 4.8 depicts the column curvature distribution as per instrumented in the test. At final loading stages, the curvature values measured at the upper column were slightly larger than those at the lower column during negative loading directions. However, the curvature at the lower column was larger than that in upper column during positive loading cycles.

4.3.1.4 Joint behavior

The evaluation of joint behavior, bond stress of beam and column reinforcement, joint shear distortion and joint shear expansion is discussed in the sections that follow.

General behavior

In the loading to a DR of 0.75%, no cracks were observed in the joint region. Initial diagonal tension and compression cracks occurred in the joint core in a loading to DR=1.0%. Maximum nominal horizontal shear stress of 6.0 MPa ($0.08 f'_c$) was observed in the joint core where f'_c is the measured compressive cylinder strength of concrete.

Bond stresses of the longitudinal beam and column bars in the joint

The average bond stresses for beam and column bars are plotted in **Figures 4.9** and **4.10**, respectively. The bond stress values were calculated at the peaks of the selected loading cycles using the wire strain gauge readings.

* All dimensions are in *mm*

Figure 4.7 Strain profiles of column bars in Specimen IN80

* All dimensions are in *mm*

Figure 4.8 Curvature distribution of columns of Specimen IN80

The average bond stresses measured along the longitudinal beam and column bars in the joint are assumed to be uniformly distributed over the gauge length. The maximum bond stress obtained along top beam bars was 1.9 MPa ($0.02 f'_c$) while the maximum bond stress along the bottom beam bars was 4.8 MPa ($0.06 f'_c$). The bond stress is observed to increase at higher drift ratios, which is an indication of a steady bond condition.

On the other hand, column reinforcing bars remained elastic during the test with maximum bond stress of 4.8 MPa ($0.06 f'_c$) on the side bar and 6.0 MPa ($0.07 f'_c$) on the center bar, respectively

* All dimensions are in *mm*

Figure 4.9 Bond stress of beam bars in Specimen IN80

Figure 4.10 Bond stress of column bars in Specimen IN80

Figure 4.11 Joint shear distortion and expansion of Specimen IN80

Joint shear distortion and expansion

The measured joint shear distortion and expansion are shown in **Figure 4.11**. The procedure of joint shear distortion and expansion estimation was described in **Chapter 3**. It is observed the joint shear distortion and expansion had small values at DR=1.0%, while they experienced a significant increase at the DR of 2.0% to 3.0%. This increase is because of the formation of diagonal tension cracks in the joint region. In the loading to DR=4.0%, joint diagonal tension cracks opened wide with the maximum joint distortion of 0.35% and the maximum joint expansion of 0.4 mm.

4.3.1.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 4.12** for Specimen IN80. The major source of the story drift was the beam displacement, indicating a “strong-column and weak-beam” response. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 21% to 61% and 2% to 8%, respectively. The contribution of beam fixed-end rotation increased gradually in the test. The contribution of column fixed-end rotation to the total drift was 14% to 23%. The maximum contribution of joint shear to the horizontal displacement was 11% at a DR of 2.0%.

Figure 4.12 Contribution of components to horizontal displacement in IN80

4.3.2 Test Results of Specimen IH80

4.3.2.1 General behavior

The final cracking pattern and the story shear force versus horizontal displacement relationship for Specimen IH80 are illustrated in **Figures 4.13** and **4.14**. The specimen reached its theoretical capacity at a DR of 3.5% in the positive loading direction and at a DR of 3.0% in the negative loading direction. Maximum capacities of 148 kN in the positive and 151 kN in the negative loading directions were reached at final loading cycles. These capacities were higher by 41% and 37% in comparison to those attained by IN80. The specimen managed to resist horizontal loading up to the drift ratio of 4.0% without any significant loss in lateral loading capacity.

The progressive cracking pattern of the specimen is shown in **Figure 4.15**. In the load cycle to DR=1.0%, flexural cracks spread along the top and bottom chord of the beams. A few fine cracks appeared in the joint panel in the load run toward DR=0.75%. The first diagonal crack in the joint occurred in the negative load run toward DR=-1.0% and was located at the left lower corner of the joint panel. When the unit was taken to a displacement equal to DR=2.0%, diagonal cracks developed from corner to corner in the joint panel. More shear-flexural cracks were observed to spread along the beam spans. By DR=3.0%, cracks in the plastic hinge region of beams widened further and the joint panel remained in the crack pattern without too much change. The crack pattern in beams was generally similar to that of Specimen IN80 but the crack width in the plastic hinge region of IH80 is considerably smaller. In addition, the number of cracks in columns and in the joint is less in Specimen IH80, which indicates the effect of using HSS reinforcements. In the load cycle to DR=4.0%, cracking in the joint region remained essentially similar to that of the previous load cycle with crack widths remaining the same. A slip of beam bars was obvious at this stage by observing that cover concrete around corners at the intersection of beams and columns were crushed, especially in the bottom corners.

Figure 4.13 Final crack pattern of Specimen IH80

Figure 4.14 Story shear force versus horizontal displacement for Specimen IH80

(a) Drift Ratio of 1.0%

(b) Drift Ratio of 2.0%

(a) Drift Ratio of 3.0%

(b) Drift Ratio of 4.0%

Figure 4.15 Progressive cracking development of Specimen IH80

Figure 4.16 Energy dissipation of Specimen IH80

Furthermore, the energy dissipation of this specimen is depicted in **Figure 4.16**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are decreasing after a DR of 3.5% for second loading cycles. However, the energy dissipation is increasing continuously during the test for the first loading cycles. The energy dissipation value at DR=4.0% in the first loading cycle is 71% higher than that in the second cycle.

4.3.2.2 Beam behavior

The evaluation of strains in beam reinforcement and the beam curvature distribution are discussed in the following sections. Strain profiles are shown for the positive and negative loading directions to DR=1.0% and DR=3.0%.

Longitudinal beam bar strains

Figure 4.17 shows the strain profiles of the top and bottom beam reinforcement of IH80. As illustrated, the beam bar in tension started to yield at the column face after the DR of 3.0% while the strain at the compression face was relatively low. This is compatible with the observed plastic hinge formation in beam ends. However, the strain gauges at the center of the joint showed the strain values lower than the yielding value throughout the test and the strain values were dropping sharply through the joint region. In addition, it was observed that the strain readings during positive loading cycles were significantly higher than those in subsequent negative loading cycles. This explained the effective bond strength of the joint core, which enables stress reversals in longitudinal beam bars.

Furthermore, strain values in Specimen IH80 in the middle of the joint core are higher than those in IN80. Bond deterioration in the joint region is observed to be more severe in this specimen with higher reinforcement grade.

Beam curvature distributions

Figure 4.18 illustrates the beam curvature distributions estimated from the transducer readings. Curvature values are presented for drift ratios of DR=1.0% and DR=3.0% in the positive and negative loading directions. It is observed that the beam curvatures measured near the column face reached the theoretical yield value after a DR of 1.0%.

* All dimensions are in *mm*

Figure 4.17 Strain profiles of beam bars in Specimen IH80

* All dimensions are in *mm*

Figure 4.18 Curvature distribution of beams of Specimen IH80

However, the beam curvatures increase was rapid in the subsequent loading cycles in both of the loading directions mainly because of the formation of plastic hinges in beam ends.

4.3.2.3 Column behavior

The evaluation of strains along the side and middle reinforcing bars in the column section as well as the column curvature distribution are discussed in the following sections. Strain profiles are shown for the positive and negative loading directions to DR=1.0% and DR=3.0%.

Longitudinal column bar strains

The strain distributions of side and center bars of columns are depicted in **Figure 4.19**. It is observed that during positive and negative loading cycles, the side reinforcements experience both tension and compression while the strain in the center bars only remains in compression. The strain values in higher drift ratios are slightly increasing. Although the strain values are still smaller than yielding values, they are higher than those in Specimen IN80 with normal-strength steel reinforcements.

Column curvature distributions

Figure 4.20 shows the column curvature distribution as per instrumented in the test. During loading cycles to DR=1.0%, the curvature values at the top and bottom columns were almost similar and they were smaller than the yielding curvature values. At higher drift ratios, the curvature values at the upper column were slightly larger than those at the lower column during the both positive and negative loading directions.

* All dimensions are in *mm*

Figure 4.19 Strain profiles of column bars in Specimen IH80

* All dimensions are in *mm*

Figure 4.20 Curvature distribution of columns of Specimen IH80

4.3.2.4 Joint behavior

The bond stress along the beam and column reinforcing bars, joint shear distortion and joint shear expansion are evaluated in the following sections to clarify the joint behavior.

General behavior

Maximum nominal horizontal shear stress of 5.4 MPa ($0.07 f'_c$) was observed in the joint core. No cracks were observed in the joint region up to the loading to a DR of 0.75%. Initial diagonal tension and compression cracks occurred in the joint core in loading to DR=1.0%.

Bond stresses of the longitudinal beam and column bars in the joint

Figures 4.21 and 4.22 demonstrate the average bond stresses along the beam and column bars, respectively. The average bond stresses measured along the longitudinal beam and column bars in the joint region are assumed to be uniformly distributed over the gauge length. The maximum bond stress obtained along top beam bars was 1.4 MPa ($0.02 f'_c$) while the maximum bond stress observed along the bottom beam bars was 3.4 MPa ($0.04 f'_c$). The bond stress along the top beam bars is observed to increase at a higher drift ratio, which is an indication of a steady bond condition. On the other hand, bond deterioration was obvious in the bottom beam bars where bond stresses decreased at the end of the test. The longitudinal column reinforcements remained elastic until the end of the test with the maximum bond stress of 2.9 MPa ($0.04 f'_c$) on the side and center bars.

Joint shear distortion and expansion

Figure 4.23 illustrates the measured joint shear distortion and expansion. It is observed the joint shear distortion and expansion had small values at DR=1.0%, while they experienced a significant increase until DRs of 2.0% to 3.0%. In the loading to DR=4.0%, the joint diagonal tension cracks opened wide with the maximum joint distortion of 0.2% and the maximum joint expansion of 0.8 mm.

* All dimensions are in *mm*

Figure 4.21 Bond stress of beam bars in Specimen IH80

* All dimensions are in *mm*

Figure 4.22 Bond stress of column bars in Specimen IH80

Figure 4.23 Joint shear distortion and expansion of Specimen IH80

4.3.2.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is depicted in **Figure 4.24** for Specimen IH80. The major source of the story drift was the beam displacement, indicating a “strong-column and weak-beam” response. The contributions of the beam fixed-end and flexure rotation to the total drift ranged from 12% to 47% and 3% to 9%, respectively. The contribution of the beam fixed-end rotation increased gradually in the test. The contribution of column fixed-end rotation also increased gradually in the test. However, the contribution of column flexural deformation did not change significantly during the test. The contribution of column fixed-end and column flexural rotation to the total drift was 12% to 25% and 2% to 4%. The maximum contribution of joint shear to the horizontal displacement was 5% at a DR of 2.0%.

Figure 4.24 Contribution of components to horizontal displacement in IH80

4.3.3 Test Results of Specimen IH80A

4.3.3.1 General behavior

The final cracking pattern and the horizontal story shear force versus horizontal displacement hysteresis loops of Specimen IH80A are shown in **Figures 4.25** and **4.26**. The specimen reached its theoretical capacity at a DR of 2.5% in the negative loading direction while it did not reach its theoretical capacity in the positive loading direction. The maximum attained capacities were 138 kN in the positive and 151 kN in the negative loading directions. These maximum capacities were reached at a DR of 3.0% in the positive loading direction and at a DR of 2.5% in the negative direction. This maximum capacity in the negative loading direction was lower by 7% in comparison to that attained by Specimen IH80 while the maximum capacity in the positive loading direction had the same value in both specimens. Due to the fracture of longitudinal beam bars at DR=4.0%, significant deterioration is observed in lateral loading capacity of the specimen.

The progressive cracking pattern of the specimen is illustrated in **Figure 4.27**. In the load cycle corresponding to DR=1.0%, flexural cracks occurred in the top and bottom beam chords during the positive and negative loading directions. No cracks were observed in columns and in the joint region. In the load cycle to DR=2.0%, more flexural cracks occurred in beam plastic hinge regions. Moreover, new diagonal cracks were observed in beams while some existing cracks changed direction and became diagonally oriented. By DR=3.0%, flexural cracks in beam plastic hinge regions continue to widen and some incipient crushing of concrete was observed in the mid-height of the left beam in an approximate distance of 130 mm from the joint edge. In addition, some horizontal cracks occurred along the beam top and bottom longitudinal bars. This indicates that the deterioration of bond condition has occurred in the joint panel. As the test proceeded towards DR=4.0%, beam bottom chords adjacent to the column were crushed significantly and large gap openings occurred in beam plastic hinge regions. Beam bottom bars were fractured at the final loading stage of the test.

Figure 4.25 Final crack pattern of Specimen IH80A

Figure 4.26 Story shear force versus horizontal displacement for Specimen IH80A

Figure 4.27 Progressive cracking development of Specimen IH80A

Figure 4.28 Energy dissipation of Specimen IH80A

Furthermore, the energy dissipation of the specimen is shown in **Figure 4.28**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are decreasing after a DR of 3.5% for the first and second loading cycles. The energy dissipation value at DR=3.5% in the first loading cycle is 15% higher than that in the second cycle.

4.3.3.2 Beam behavior

The beam behavior is discussed via the evaluation of strains in beam reinforcement and beam curvatures:

Longitudinal beam bar strains

Figure 4.29 explains the strain profiles of the top and bottom beam reinforcement of Specimen IH80A. At DR=1.0%, beam reinforcing bars were in elastic range where none of the strain gauges exceeded the yield strain limit. Plastic hinges were formed at beam sections close to the column face when DR=3.0% was attained. As a result, large strain values were recorded along the longitudinal beam bars at the plastic hinge region at a DR of 3.0%.

Beam curvature distributions

Figure 4.30 shows the beam curvature distributions estimated from transducer readings. The beam curvature measured near the column face almost reached the theoretical yield curvature in the loading stage of DR=1.0%. Beam curvature increased rapidly after a DR of 3.0% and exceeded its theoretical yield curvature. This observation is compatible with the observed beam reinforcement strains.

4.3.3.3 Column behavior

The column behavior is evaluated based on the strain values measured in column side and middle bars. The column curvature was obtained from the measured displacement transducers along the column.

Longitudinal column bar strains

The strain distributions of side and center bars in the column section are explained in **Figure 4.31**. Due to the confinement effect of column axial compression loading, column bars basically remained elastic throughout the test. Although the strain of column bars increased gradually, they did not reach the yield strain.

* All dimensions are in *mm*

Figure 4.29 Strain profiles of beam bars in Specimen IH80A

* All dimensions are in *mm*

Figure 4.30 Curvature distribution of beams of Specimen IH80A

Column curvature distributions

Figure 4.32 shows the column curvature distributions as per instrumented in the test. The measured curvatures of upper column were similar to those at the lower column. As the column was still in its elastic range, respective curvatures increased gradually but no rapid increase was observed.

4.3.3.4 Joint behavior

The bond stress of beam and column reinforcing bars, joint shear distortion and joint shear expansion are discussed in the sections that follow.

General behavior

In Specimen IH80A, the maximum nominal horizontal shear stress in the joint core was 7.6 MPa ($0.09 f'_c$). As illustrated in **Figure 4.27**, no diagonal cracking was observed in the joint region as a result of the presence of the column axial loading.

Bond stresses of the longitudinal beam and column bars in the joint

Figures 4.33 and **4.34** show the average bond stresses along the beam and column bars, respectively. The maximum bond stress obtained along top beam bars was 2.4 MPa ($0.03 f'_c$), while the maximum bond stress observed along the bottom beam bars was 5.6 MPa ($0.07 f'_c$). Bond stress values increased in higher drift ratios, which is an indication of satisfactory bond behavior. On the other hand, the column reinforcement remained elastic until the end of the test with the maximum bond stress of 1.1 MPa ($0.01 f'_c$) along side bars and maximum bond stress of 0.5 MPa ($0.006 f'_c$) along center bars, respectively.

Joint shear distortion and expansion

Figure 4.35 explains the measured joint shear distortion and expansion of IH80A. The observed values are generally small because of the beneficial confinement effects of column axial compression loading. In the final loading to DR=4.0%, the maximum joint distortion is 0.04% and the maximum joint expansion is 0.09 mm.

* All dimensions are in *mm*

Figure 4.31 Strain profiles of column bars in Specimen IH80A

* All dimensions are in *mm*

Figure 4.32 Curvature distribution of columns of Specimen IH80A

* All dimensions are in *mm*

Figure 4.33 Bond stress of beam bars in Specimen IH80A

* All dimensions are in *mm*

Figure 4.34 Bond stress of column bars in Specimen IH80A

Figure 4.35 Joint shear distortion and expansion of Specimen IH80A

4.3.3.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is depicted in **Figure 4.36** for Specimen IH80A. The major source of the story drift was the beam displacement. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 18% to 50% and 15% to 16%, respectively. The contribution of beam fixed-end rotation increased gradually, while the contribution of beam flexural rotation did not change during the test. In addition, the contribution of column fixed-end rotation also increased gradually in the test. However, the contribution of column flexural deformation did not change significantly during the test. The contribution of column fixed-end and column flexural rotation to the total drift was 3% to 25% and 0.2% to 0.6%. The maximum contribution of joint shear to the horizontal displacement was 1% at a DR of 1.0%. The contribution of joint shear distortion did not increase and remained negligible during the test.

Figure 4.36 Contribution of components to horizontal displacement in IH80A

4.4 Test Result of Group Two (IN100 and IH100)

Group two consists of specimens IN100 and IH100 with same dimensions as described in **Chapter 3**. Specimen IN100 was an HSC beam-column joint with normal-strength steel reinforcement while IH100 was reinforced with high-strength steel. Both of the specimens were tested without column axial loading.

The typical features of these specimens are summarized as follows:

- Smallest diameter of top and bottom bars in the beam section = 16 mm
- Actual maximum beam-bar to column-depth ratio = 1/28
- Actual column-bar to beam-depth ratio = 1/25
- Calculated maximum beam-bar to column-depth ratio based on Eq. 9-21 in NZS 3101-06 [N3]:

$$\text{IN100} = 1/22 \qquad \text{IH100} = 1/31$$
- Calculated maximum column-bar to beam-depth ratio based on clause 10.4.6.6 in NZS 3101-06 [N3]:

$$\text{IN100} = 1/16 \qquad \text{IH100} = 1/22$$
- Specimen IN100 met both the requirement of maximum beam-bar to column-depth and maximum column-bar to beam-depth set in NZS 3101-06 [N3], whereas IH100 did not satisfy the requirement of maximum beam-bar to column-depth ratio.
- Joint core reinforcement detailing complied with the requirement in clause 15.4.4.4 in NZS 3101-06 [N3] (maximum bar spacing = 65 mm, allowable bar spacing = 160 mm)

The following sections report the test result of Specimens IN100 and IH100.

4.4.1 Test Results of Specimen IN100

4.4.1.1 General behavior

The final cracking pattern of Specimen IN100 and the story shear force versus horizontal displacement relationship are shown in **Figures 4.37** and **4.38**, respectively. The specimen reached its theoretical capacity at a DR of 2.0% in the both negative and positive loading directions. The maximum capacity of 108 kN was reached at the DR of 2.0% and 3.0% in the positive and negative loading directions, respectively. At DR=4.0%, the specimen experienced a substantial loss

of 20% of its load-carrying capacity in the positive loading direction. Up to the final loading cycle, the residual load-carrying capacity in the positive and negative loading direction was 70 kN and 86 kN, which is approximately 67% and 83% of the theoretical capacities.

The cracking pattern of Specimen IN100 in different drift ratios are depicted in **Figure 4.39**. In the load cycle to DR=1.0%, long cracks vertically propagated from beam webs toward beam compression zone. Most of vertical cracks in the beam web propagating from top and bottom chords merged in the reversed load run. Cracks in columns were limited to approximately three cracks forming on each tension side of columns including one located at the beam face. Several diagonal cracks developed in the joint region in this step. By DR=2.0%, more shear-flexural cracks were observed to spread along beams away from column faces especially on the beam top chords. Additional joint diagonal cracks occurred in the both positive and negative loading directions. Several additional flexural cracks of small width occurred in columns at this stage. By DR=3.0%, flexural cracks in beam plastic hinge regions continued to widen and the joint panel remained in the crack pattern without too much change. No change of cracking appearance in columns was observed. As the test proceeded to load cycle of DR=4.0%, crushing of concrete occurred on the top and bottom beam chords in plastic hinge regions. However, the damage of concrete was more severe on beam bottom chords, which spread further toward the center of the beam height. Longitudinal beam bottom bars were exposed at this stage.

Furthermore, the energy dissipation of the specimen is illustrated in **Figure 4.40**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are decreasing after a DR of 3.5% for the first and second loading cycles. The energy dissipation value at DR=4.0% in the first loading cycle is 29% higher than that in the second cycle.

4.4.1.2 Beam behavior

The evaluation of strain distribution along beam bars as well as beam curvatures are discussed in the following sections. Strain profiles are shown for pushing and pulling loading directions to DR=1.0% and DR=3.0%.

Figure 4.37 Final crack pattern of Specimen IN100

Figure 4.38 Story shear force versus horizontal displacement for Specimen IN100

(a) Drift Ratio of 1.0%

(b) Drift Ratio of 2.0%

(a) Drift Ratio of 3.0%

(b) Drift Ratio of 4.0%

Figure 4.39 Progressive cracking development of Specimen IN100

Figure 4.40 Energy dissipation of Specimen IN100

Longitudinal beam bar strains

The strain profiles of top and bottom beam reinforcing bars of Specimen IN80 are explained in **Figure 4.41**. Top beam bars did not reach their yielding point while bottom bars yielded at the column face at DR=1.0%. As illustrated, the strain gauges at the joint center showed strain values smaller than the yielding strain value throughout the test. However, the compressive strain along beam bars had much smaller values than the tension strain.

Beam curvature distributions

Figure 4.42 depicts the beam curvature distributions estimated from transducer readings. During DR=1.0%, curvature values almost reached the theoretical yield curvature. At a DR of 3.0%, plastic hinges are formed in beams and a sharp increase in beam curvature values is observed at column faces.

4.4.1.3 Column behavior

The evaluation of strains in side and middle reinforcing bars in columns as well as the column curvature distribution are discussed in the following sections. The figures are presented based on strain and curvature profiles at DRs of 1.0% and 3.0% during positive and negative loading cycles.

Longitudinal column bar strains

Figure 4.43 shows the strain distribution along the side and center bars of columns. As a general observation, strain values did not reach the yield strain throughout the test. Side bars experienced both tension and compression strains during the pushing and pulling cycles whereas the center bars were under compression throughout the test.

Column curvature distributions

The column curvature distributions as per instrumented in the test is explained in **Figure 4.44**. The curvatures measured at the lower column were slightly larger than those at the upper column. Curvature values were smaller than yield curvature in a DR of 1.0%. Although it was evident that the column remained elastic during the test, sharp increase in curvature values are observed in DR=3.0%. This observation may not be indicative as the curvature values are based on LVDT readings, which are not accurate anymore after concrete crushing in plastic hinge regions.

* All dimensions are in *mm*

Figure 4.41 Strain profiles of beam bars in Specimen IN100

* All dimensions are in *mm*

Figure 4.42 Curvature distribution of beams of Specimen IN100

* All dimensions are in *mm*

Figure 4.43 Strain profiles of column bars in Specimen IN100

* All dimensions are in *mm*

Figure 4.44 Curvature distribution of columns of Specimen IN100

4.4.1.4 Joint behavior

The joint behavior on bond stress of beam and column reinforcement, joint shear distortion and joint shear expansion are discussed in the sections that follow.

General behavior

Diagonal tension cracks were initiated in the DR of 1.0% and more cracks appeared in the joint core in the subsequent loading cycles, but they did not widen during the test. The maximum nominal horizontal shear stress in the joint core was 4.1 MPa or $0.04 f'_c$, which is smaller than those in Specimens IN80 and IH80.

Bond stresses of the longitudinal beam and column bars in the joint

The average bond stresses were measured along the longitudinal beam and column bars in the joint region. As plotted in **Figures 4.45** and **4.46**, stress values were calculated based on wire strain gauge readings assuming that the stress was uniformly distributed over the gauge length. In top beam bars, the maximum bond stress was found to be 1.3 MPa ($0.01 f'_c$), while the maximum bond stress observed along the bottom beam bars was 6.3 MPa ($0.06 f'_c$). Bond stress values along the top bars were close to those in Specimens IN80 and IH80, while the bond stress along bottom bars were noticeably larger than those other specimens. In addition, bond deterioration was obvious in the bottom beam bars where bond stresses decreased rapidly at the end of test.

On the other hand, the column reinforcement remained elastic until the end of the test with the maximum bond stress of 3.3 MPa ($0.03 f'_c$) on side bars and a maximum bond stress of 4.0 MPa ($0.04 f'_c$) on center bars, respectively.

Joint shear distortion and expansion

The measured joint shear distortion and expansion are shown in **Figure 4.47**. There was a constant increase in joint shear distortion and expansion only until a DR of 3.0%. In subsequent loading cycles to DR=4.0%, the joint distortion and expansion slightly decreased and the maximum joint distortion was 0.4% and the maximum joint expansion was 0.8 mm at the end of the test, respectively.

* All dimensions are in *mm*

Figure 4.45 Bond stress of beam bars in Specimen IN100

* All dimensions are in *mm*

Figure 4.46 Bond stress of column bars in Specimen IN100

Figure 4.47 Joint shear distortion and expansion of Specimen IN100

4.4.1.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 4.48** for Specimen IN100. The major source of the story drift was the beam displacement. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 23% to 53% and 2% to 10%, respectively. The contribution of beam fixed-end rotation increased gradually, while the contribution of beam flexural rotation decreased significantly at the final loading cycle. In addition, the contribution of column fixed-end rotation did not change during the test. However, the contribution of column flexural deformation increased slightly at the end of the test. The contribution of column fixed-end and column flexural rotation to the total drift was 16% to 25% and 1% to 3%. The maximum contribution of joint shear to the displacement was 10% at a DR of 1.0%. The contribution of joint shear distortion decreased slightly in subsequent loading cycles.

Figure 4.48 Contribution of components to horizontal displacement in IN100

4.4.2 Test Results of Specimen IH100

4.4.2.1 General behavior

Figures 4.49 and **4.50** show the final cracking patterns of Specimen IH100 and the measured story shear force versus horizontal displacement hysteresis loops. The specimen did not reach its theoretical capacity till the end of the test. The maximum attained capacities were 141 kN in the positive and 147 kN in the negative loading directions, which were approximately 93% and 97% of the theoretical capacity. These maximum capacities were corresponding to a DR of 3.5% in the both loading directions. These capacities were higher by 31% and 36% in comparison to those attained by Specimen IN100 in the respective loading directions. Up to the final loading cycle, the residual load-carrying capacity in both loading directions was 124 kN, which is approximately 82% of the theoretical capacity.

The progressive cracking pattern is illustrated in **Figure 4.51**. During the initial loading cycles to DR=1.0%, the cracking pattern in beams, columns and the joint was generally similar to that of Specimen IN100. Fine cracks with small width were observed in the joint region during the loading cycle to a DR of 0.5% and diagonal cracks were observed at DR=1.0%. Flexural cracks occurred in beams and columns in positive and negative loading cycles at this stage. As the test proceeded to DR=2.0%, cracking in the joint region became much denser and the shear-flexural cracks in beam propagated rapidly, covering nearly the entire length of the beam span. Few more flexural cracks also occurred on column faces. In load cycles to DR=3.0%, no additional cracks were observed on column faces neither in the joint. However, flexural cracks in beam plastic hinge region continued to widen. By DR=4.0%, spalling of the cover concrete is observed on the beam bottom chord. However, damage of concrete in Specimen IH100 is considerably less severe compared to that in Specimen IN100, making the influence of utilizing HSS longitudinal reinforcements more evident.

Figure 4.49 Final crack pattern of Specimen IH100

Figure 4.50 Story shear force versus horizontal displacement for Specimen IH100

(a) Drift Ratio of 1.0%

(b) Drift Ratio of 2.0%

(a) Drift Ratio of 3.0%

(b) Drift Ratio of 4.0%

Figure 4.51 Progressive cracking development of Specimen IH100

Figure 4.52 Energy dissipation of Specimen IH100

Furthermore, energy dissipation capacity of the specimen is shown in **Figure 4.52**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are increasing continuously during the test for the first and second loading cycles. The energy dissipation value at DR=4.0% in the first loading cycle is 35% higher than that in the second cycle.

4.4.2.2 Beam behavior

The following section demonstrates the evaluation of strains in beam reinforcement as well as beam curvature distributions.

Longitudinal beam bar strains

Figure 4.53 depicts the strain profiles of the top and bottom beam reinforcement of Specimen IH100. Yielding of top and bottom beam bars was not noticed until the DR of 3.0%. For higher-grade longitudinal bars in IH100, strain values in the middle of the joint core are higher than those in IN100. Bond deterioration in the joint region is observed to be more severe in the specimens with higher reinforcement grades.

Beam curvature distributions

Figure 4.54 illustrates beam curvature distributions which were estimated based on transducer readings. Beam curvature values were below the theoretical yield curvature at DR=1.0%. After a DR of 3.0%, measured beam curvatures were above the yielding values with the plastic hinge development at beam ends facing the column.

4.4.2.3 Column behavior

The strain distribution in column side and center bars and column curvatures are addressed as below.

Longitudinal column bar strains

As explained in **Figure 4.55**, the gauged column reinforcement shows no sign of yielding where the measured strains are well below its yielding strain limit. As the horizontal lateral loading was increased, the strains increased gradually but the reinforcing bar was elastic when the test reached the maximum stage.

* All dimensions are in *mm*

Figure 4.53 Strain profiles of beam bars in Specimen IH100

* All dimensions are in *mm*

Figure 4.54 Curvature distribution of beams of Specimen IH100

* All dimensions are in *mm*

Figure 4.55 Strain profiles of column bars in Specimen IH100

* All dimensions are in *mm*

Figure 4.56 Curvature distribution of columns of Specimen IH100

Column curvature distributions

Figure 4.56 shows column curvature distributions based on the readings of LVDTs which were installed along the column. The curvatures measured at the upper column were almost identical to that in the lower column.

4.4.2.4 Joint behavior

The discussion on joint behavior in the following sections covers the evaluation of bond stress of beam and column reinforcement, joint shear distortion and joint shear expansion.

General behavior

As shown in **Figure 4.51**, initial diagonal tension cracks were observed during the DR of 1.0%. In subsequent loading cycles, more cracks appeared on the joint core and propagated rapidly, but they did not widen. Maximum nominal horizontal shear stress of 5.5 MPa ($0.05 f'_c$) was observed.

Bond stresses of the longitudinal beam and column bars in the joint

The average bond stresses obtained for beam bars and column bars are plotted in **Figures 4.57** and **4.58**. The maximum bond stresses along the top and bottom beam bars were 2.0 MPa ($0.02 f'_c$) and 5.3 MPa ($0.05 f'_c$), respectively. On the other hand, the column reinforcement remained elastic throughout the test with maximum bond stress of 7.1 MPa ($0.07 f'_c$) on the side bar and maximum bond stress of 2.6 MPa ($0.03 f'_c$) on the center bar.

Joint shear distortion and expansion

Figure 4.59 depicts the measured joint shear distortion and expansion based on instrumentation in the joint core. Generally, the development of joint shear distortion and joint shear expansion was rapid in the test from DR=1.0% to DR=4.0%. The maximum joint distortion was 0.4% and the maximum joint expansion was 1.3 mm.

Figure 4.57 Bond stress of beam bars in Specimen IH100

* All dimensions are in *mm*

Figure 4.58 Bond stress of column bars in Specimen IH100

Figure 4.59 Joint shear distortion and expansion of Specimen IH100

4.4.2.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 4.60** for Specimen IH100. The major source of the story drift was the beam displacement. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 18% to 43% and 12% to 15%, respectively. The contribution of beam fixed-end rotation increased gradually, while the contribution of beam flexural rotation did not change significantly during the test. In addition, the contribution of column fixed-end rotation increased significantly at the final loading cycle. However, the contribution of column flexural deformation remained almost constant during the test. The contribution of column fixed-end and column flexural rotation to the total drift was 14% to 25% and 1% to 3%. The maximum contribution of joint shear to the horizontal displacement was 9% at a DR of 2.0%. The contribution of joint shear distortion did not alter significantly during the test.

Figure 4.60 Contribution of components to horizontal displacement in IH100

4.5 Test Result of Group Three (IH60 and IH60A)

Group three consists of specimens IH60 and IH60A with same dimension as described in **Chapter 3**. Both of the specimens were HSC beam-column joints with HSS reinforcement. IH60 was tested without column axial loading while Specimen IH60A was tested with column axial compression loading of $0.3 f'_c A_g$.

The typical features of these specimens are summarized as follows:

- Smallest diameter of top and bottom bars in the beam section = 19 mm
- Actual maximum beam-bar to column-depth ratio = 1/24
- Actual column-bar to beam-depth ratio = 1/25
- Calculated maximum beam-bar to column-depth ratio based on Eq. 9-21 in NZS 3101-06 [N3]:

$$\text{IH60} = 1/36 \qquad \text{IH60A} = 1/31$$
- Calculated maximum column-bar to beam-depth ratio based on clause 10.4.6.6 in NZS 3101-06 [N3]:

$$\text{IH60} = 1/28 \qquad \text{IH60A} = 1/28$$
- Both of the specimens met the both requirements of maximum beam-bar to column-depth and maximum column-bar to beam-depth ratios as set in NZS 3101-06 [N3].
- Joint core reinforcement detailing complied with the requirement in clause 15.4.4.4 in NZS 3101-06 [N3] (maximum bar spacing = 85 mm, allowable bar spacing = 200 mm)

The following sections report the test result of Specimens IH60 and IH60A.

4.5.1 Test Results of Specimen IH60

4.5.1.1 General behavior

Figures 4.61 and **4.62** show the final cracking pattern of Specimen IH60 as well as the measured horizontal story shear force versus horizontal displacement hysteresis loops, respectively. The specimen almost reached its theoretical capacity in the negative loading direction. However, the bond-slip of the rebars resulted in lower story shear forces in the positive direction. The maximum attained capacities were 112 kN in the positive and 127 kN in the negative loading directions, which were

approximately 82% and 93% of theoretical capacities. These maximum capacities were reached at a DR of 3.0% in both loading directions. Up to the final loading cycle, the residual load-carrying capacity was 79 kN in positive and 106 kN in the negative loading direction, which was approximately 58% and 78% of the theoretical capacity.

The progressive cracking pattern is shown in **Figure 4.63**. In the load cycle to DR=1.0%, flexural cracks in the positive and negative loading directions propagated in a vertically upward or downward direction covering full depth of the beam and spreading along half of the beam span. There was no shear crack observed on the beam portion. Several diagonal cracks developed in the joint region at this stage, while no cracks were developed on column faces. By DR=2.0%, shear-flexural cracks appeared along the beam span and more diagonal cracks occurred in the joint region in the positive loading direction. In loading runs to DR=3.0%, some more fine cracks developed in the joint panel. In addition, shear cracks occurred in the east and west beam. Incipient crushing of cover concrete on the top and bottom chord of the west beam at the column face was observed; indicating that the bond of beam bars in the joint region had started to deteriorate. As the test proceeded toward DR=4.0%, the cracking pattern in beams and in the joint region was similar to that of previous loading cycles. Crush of concrete in the four corners and split of cover concrete along the top and bottom beam bars, which are all indications of bond slip were in progress. Crushing of concrete on the top and bottom beam chords adjacent to column faces spread along beams away from column faces. No cracks were observed in the top and bottom column throughout the test.

Furthermore, the energy dissipation of specimen is shown in **Figure 4.64**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are increasing continuously during the test for the first and second loading cycles. The energy dissipation value at DR=4.0% in the first loading cycle is 22% higher than that in the second cycle.

4.5.1.2 Beam behavior

The strain profiles of beam reinforcement and also the beam curvatures are explained in the sections that follow.

Figure 4.61 Final crack pattern of Specimen IH60

Figure 4.62 Story shear force versus horizontal displacement for Specimen IH60

(a) Drift Ratio of 1.0%

(b) Drift Ratio of 2.0%

(a) Drift Ratio of 3.0%

(b) Drift Ratio of 4.0%

Figure 4.63 Progressive cracking development of Specimen IH60

Figure 4.64 Energy dissipation of Specimen IH60

Longitudinal beam bar strains

As illustrated in **Figure 4.65**, top and bottom beam bars in Specimen IH60 showed a sign of yielding as the strain gauges installed near the column face exceeded the yield strain. When the DR of 3.0% was attained, yielding of beam reinforcement was observed in more strain gauges confirming the propagation of bar yielding. Yield penetration was noticed where yield strain was penetrated through a certain distance into the joint core. Despite the use of large diameter beam reinforcement, the beam bars within the joint core demonstrated good bonding as the tensile stress on one face was transferred to compressive stress at another face.

Beam curvature distributions

The beam curvature distributions estimated from the transducer readings are depicted in **Figure 4.66**. Due to plastic hinge formation in the beam ends at DR=1.0%, the measured beam curvature at the column face exceeded the theoretical yield curvature. A rapid increase of the curvature was observed in the subsequent loading cycles, which shows the high energy dissipation of beams.

4.5.1.3 Column behavior

The evaluation of strains in column reinforcement and column curvatures are discussed in the sections that follow.

Longitudinal column bar strains

Generally, the strain values along the column bars did not reach the yield strain as explained in **Figure 4.67**. The column side bars experienced a huge variation between the compression and tension strain, while the center bars experienced only the tension strain during the test.

Column curvature distributions

Column curvature distributions as per instrumented in the test are explained in **Figure 4.68**. Curvature values were smaller than the yield curvature in a DR of 1.0%. Although it was evident that the column remained elastic during the test, an increase of curvature values is observed in DR=3.0%.

* All dimensions are in *mm*

Figure 4.65 Strain profiles of beam bars in Specimen IH60

* All dimensions are in *mm*

Figure 4.66 Curvature distribution of beams of Specimen IH60

* All dimensions are in *mm*

Figure 4.67 Strain profiles of column bars in Specimen IH60

* All dimensions are in *mm*

Figure 4.68 Curvature distribution of columns of Specimen IH60

4.5.1.4 Joint behavior

The evaluation of joint behavior on bond stress of beam and column reinforcing bars, joint shear distortion and joint shear expansion are discussed in the sections that follow.

General behavior

The diagonal tension cracks were firstly observed in a DR of 1.0% as explained in **Figure 4.63**. The cracks were propagated in subsequent loading cycles. The maximum nominal horizontal shear stress in the joint core was 6.2 MPa ($0.10 f'_c$).

Bond stresses of the longitudinal beam and column bars in the joint

The average bond stresses measured along the longitudinal beam and column bars in the joint were calculated using the wire strain gauge readings. These calculated bond stresses obtained are plotted in **Figures 4.69** and **4.70** for the beam and column bars.

The maximum bond stress along the top beam bars was 10.7 MPa ($0.17 f'_c$), while the maximum bond stress observed along the bottom beam bars was 5.6 MPa ($0.09 f'_c$). The bond behavior of the specimen was satisfactory throughout the test as the bond values increased in larger drift ratios. As for the column bars, the maximum bond stress of 2.4 MPa ($0.04 f'_c$) was observed.

Joint shear distortion and expansion

Figure 4.71 illustrates the measured joint shear distortion and expansion. It is observed the joint shear distortion and expansion had small values at DR=1.0%, while they experienced a noticeable increase up to DRs of 2.0% and 3.0%. In the loading to DR=4.0%, the joint diagonal tension cracks opened wide with the maximum joint distortion of 0.2% and the maximum joint expansion of 0.9 mm.

* All dimensions are in *mm*

Figure 4.69 Bond stress of beam bars in Specimen IH60

* All dimensions are in *mm*

Figure 4.70 Bond stress of column bars in Specimen IH60

Figure 4.71 Joint shear distortion and expansion of Specimen IH60

4.5.1.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 4.72** for Specimen IH60. The major source of the story drift was the beam displacement. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 17% to 52% and 6% to 9%, respectively. The contribution of beam fixed-end rotation increased gradually, while the contribution of beam flexural rotation decreased slightly during the test. In addition, the contribution of column fixed-end rotation increased gradually in higher drift ratios. However, the contribution of column flexural deformation decreased at the end of the test. The contribution of column fixed-end and column flexural rotation to the total drift was 14% to 25% and 0.5% to 3%. The maximum contribution of joint shear to the horizontal displacement was 7% at a DR of 1.0%. The contribution of joint shear distortion decreased slightly in subsequent loading cycles.

Figure 4.72 Contribution of components to horizontal displacement in IH60

4.5.2 Test Results of Specimen IH60A

4.5.2.1 General behavior

The final cracking pattern of Specimen IH60A and the horizontal story shear force versus horizontal displacement hysteresis loops are shown in **Figures 4.73** and **4.74**. Similar to Specimen IH60, this specimen also did not reach its theoretical capacity till the end of the test. The maximum attained capacities were 105 kN in the positive and 119 kN in the negative loading directions, which were approximately 77% and 87% of the theoretical capacities. These maximum capacities were reached at a DR of 2.0% in the positive loading direction and at a DR of 2.5% in the other direction. These capacities were lower by 6% in comparison to those attained by Specimen IH60. Up to the final loading cycle, the residual load-carrying capacity was 73 kN in positive and 99 kN in the negative loading direction, which was approximately 53% and 73% of the theoretical capacity.

The cracking development in Specimen IH60A is illustrates in **Figure 4.75**. In the load cycle to DR=1.0%, flexural cracks spread along the top and bottom chord of the beams and no cracks were observed in columns and in the joint panel. By DR=2.0%, plastic hinges developed in beams with more flexural cracks forming and widening of existing cracks. Fine cracks developed in the joint panel at this stage. In the loading run to DR=3.0%, some shear cracks occurred in beams and the cracking pattern in the joint remained unchanged. Cover concrete around the corner at the intersection of the left beam and the column began to crush at this stage. In the load cycle to DR=4.0%, cover concrete at the top and bottom beam chords next to the column faces spalled off owing to the significant slip of longitudinal bars. Moreover, cracks in beams along column faces were growing up as two big gaps. No new cracks or widening of existing cracks were observed in the column or the joint region.

Furthermore, the energy dissipation of the specimen is shown in **Figure 4.76**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are decreasing after a DR of 3.0% for the first and second loading cycles. The energy dissipation value at DR=4.0% in the first loading cycle is 29% higher than that in the second cycle.

Figure 4.73 Final crack pattern of Specimen IH60A

Figure 4.74 Story shear force versus horizontal displacement for Specimen IH60A

(a) Drift Ratio of 1.0%

(b) Drift Ratio of 2.0%

(a) Drift Ratio of 3.0%

(b) Drift Ratio of 4.0%

Figure 4.75 Progressive cracking development of Specimen IH60A

Figure 4.76 Energy dissipation of Specimen IH60A

4.5.2.2 Beam behavior

The following sections discuss the evaluation of strains in beam reinforcement as well as the beam curvatures.

Longitudinal beam bar strains

As explained in **Figure 4.77**, yielding is not observed along the top and bottom beam reinforcements at DR=1.0%. However, the strain of reinforcements near the column face experienced high values in the lateral loading to DR 3.0%. The strain gauges at the tension face showed the yielding value while the strain at the compression face was relatively low. Strain gauges installed at the middle of the joint core showed low strain readings, which explained the presence of good bond in the joint core. This is mainly because of the confinement effect caused by the applied column axial compression loading.

Beam curvature distributions

Figure 4.78 shows beam curvature distributions estimated from the transducer readings. In the early loading stage of DR 1.0%, the beam curvature measured near the column face almost reached the theoretical yield value. In subsequent loading cycles, the curvature values surpassed its yield curvature limit. Similar trend was also observed in negative loading cycles as the plastic hinges formed in beam ends.

4.5.2.3 Column behavior

The evaluation of strains in column side and center bars as well as column curvatures are discussed in the sections that follow.

Longitudinal column bar strains

As shown in **Figure 4.79**, the column basically remained elastic throughout the test as the strains of column bar did not reach the yield strain. This observation is a result of the confinement effect of the column axial compression loading. Strain values in this specimen are much smaller than those in Specimen IH60.

Column curvature distributions

Figure 4.80 shows the column curvature distributions. The column was in its elastic range at a DR of 1.0% and the respective curvatures increased gradually until the late stage loading of DR=3.0%.

* All dimensions are in *mm*

Figure 4.77 Strain profiles of beam bars in Specimen IH60A

* All dimensions are in *mm*

Figure 4.78 Curvature distribution of beams of Specimen IH60A

* All dimensions are in *mm*

Figure 4.79 Strain profiles of column bars in Specimen IH60A

* All dimensions are in *mm*

Figure 4.80 Curvature distribution of columns of Specimen IH60A

4.5.2.4 Joint behavior

The joint behavior discussion shall cover bond stress of beam and column reinforcement, joint shear distortion and joint shear expansion:

General behavior

Due to the confinement effect of the applied column axial compression loading, only few limited cracks were found on the joint core of IH60A throughout the test. The maximum nominal horizontal shear stress of 6.3 MPa ($0.10 f'_c$) was observed.

Bond stresses of the longitudinal beam and column bars in the joint

The average bond stresses for beam and column bars are shown in **Figures 4.81** and **4.82**, respectively. The maximum bond stress obtained along top beam bars was 6.1 MPa ($0.10 f'_c$), while the maximum bond stress observed along the bottom beam bars was 4.5 MPa ($0.07 f'_c$). Bond stress values increased in higher drift ratios, which is an indication of the satisfactory bond behavior. On the other hand, the maximum bond stress of 1.5 MPa ($0.02 f'_c$) and 2.6 MPa ($0.04 f'_c$) were observed on column side and center bars.

Joint shear distortion and expansion

The measured joint shear distortion and expansion are shown in **Figure 4.83**. It was observed that the joint shear distortion and expansion values remained relatively small during the test. Due to the column axial loading, the joint panel remained elastic during the test and negligible joint deformation was observed.

* All dimensions are in *mm*

Figure 4.81 Bond stress of beam bars in Specimen IH60A

* All dimensions are in *mm*

Figure 4.82 Bond stress of column bars in Specimen IH60A

Figure 4.83 Joint shear distortion and expansion of Specimen IH60A

4.5.2.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 4.84** for Specimen IH60A. The major source of the story drift was the beam displacement. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 15% to 36% and 24% to 29%, respectively. Due to the presence of column axial loading, the contribution of beam flexural rotation was significantly higher than that in Specimen IH60. In addition, the contribution of column fixed-end rotation increased slightly during the test and the contribution of column flexural rotation was considerably low. The contribution of column fixed-end rotation to the total drift was 7% to 25%. The minimum contribution to the horizontal displacement belongs to the joint shear distortion, which remained consistently low during the test.

Figure 4.84 Contribution of components to horizontal displacement in IH60A

4.6 Conclusions

The test result of seven interior beam-column joints constructed with high-strength materials was presented in this chapter. The specimens were divided into three groups based on their reinforcement detailing. According to the test results and observation, following conclusions are reached:

1. The performance of HSC interior specimens found to be satisfactory until the DR of 4.0%. Although some specimens did not meet the bond development requirement specified by NZS3101-06 [N3], the energy dissipation capacity of all the specimens was acceptable according to ACI T1.1-02 [A2].
2. The strain profile of the steel reinforcement showed that the failure mode of the specimens is controlled by the formation of plastic hinges in beam ends facing the column. This observation is consistent with the strong-column and weak-beam design philosophy.
3. During the test, flexural cracks occurred in beams and columns. However, the cracks in the column section did not widen until the end of the test. In addition, limited diagonal tension cracking was observed in the joint panel.
4. In general, the presence of axial compression loading enhanced the energy dissipation capacity of the specimens. However, no correlation was observed between the axial column loading and the maximum load attained by the specimens.
5. It was observed that using HSC coupled with HSS reinforcements improves the bond condition of the specimens. In addition, the column axial loading proved to have enhancing effects on the bond strength.

CHAPTER 5

EXPERIMENTAL RESULTS FOR EXTERIOR JOINTS

5.1 Introduction

This chapter presents the test results for each exterior joint specimen so as to reflect the behavior of specimens under reversed cyclic loading. The exterior specimens have been divided into two groups according to their reinforcement detailing as follows:

- Group four: Specimens EN80, EH80, and EH80A
- Group five: Specimens EH60 and EH60A

The test results were obtained from the strain gauge readings and instruments which were explained in **Chapter 3**. The general behavior of the specimens is presented based on the observed cracking pattern, the measured load-displacement hysteresis response, the yielding profiles and the curvature values in beams and columns.

5.2 Test Result of Group Four (EN80, EH80 and EH80A)

Group four consists of specimens EN80, EH80 and EH80A with same dimension as described in **Chapter 3**. Specimen EN80 was a HSC beam-column joint with normal-strength steel reinforcement tested without column axial compression loading. On the other hand, EH80 was reinforced with high-strength steel (HSS). Specimen EH80A had the same design detailing and configuration as EH80 but was tested under column axial compression loading of $0.3 f'_c A_g$.

The typical features of these specimens are summarized as follows:

- Smallest diameter of top and bottom bars in the beam section = 16 mm
- Actual development length of longitudinal beam bars (d_b/l_{dh}) = 1/24
- Column-bar to beam-depth ratio:

$$\text{EN80} = 1/25 \qquad \text{EH80, EH80A} = 1/22$$
- Calculated development length of longitudinal beam bars (d_b/l_{dh}) based on ACI318-11 :

$$\text{EN80} = 1/10 \quad \text{EH80, EH80A} = 1/14$$

- Calculated maximum column-bar to beam-depth based on clause 10.4.6.6 in NZS 3101-06 [N3]:

$$\text{EN80} = 1/17 \quad \text{EH80, EH80A} = 1/24$$

- Joint core reinforcement detailing complied with the requirement in clause 15.4.4.4 in NZS 3101-06 [N3] (maximum bar spacing = 65 mm, allowable bar spacing = 160 mm)

The following sections report the test result of Specimens EN80, EH80 and EH80A.

5.2.1 Test Results of Specimen EN80

5.2.1.1 General behavior

The final cracking pattern of Specimen EN80 and the story shear force versus horizontal displacement relationship are shown in **Figures 5.1** and **5.2**. In addition, the theoretical maximum story shear force P_y calculated based on the development of the beam plastic hinge has been illustrated in **Figure 5.2**. The specimen reached its theoretical capacity at DRs of 0.5% and 2.0% in the positive and negative loading directions, respectively. Maximum capacities of 44 kN in the positive and 83 kN in the negative loading directions were reached. This specimen did not experience any substantial loss of more than 20% of its load-carrying capacity until the final loading cycle.

Flexural cracks appeared corresponding to a DR of 0.5% as presented in **Figure 5.3**. In subsequent loading cycles, flexural cracks in the positive and negative loading directions propagated in a vertically upward or downward direction covering full depth of the beam and spreading along the beam span. In the load cycle to DR=2.0%, some cracks changed direction and became diagonally oriented when they penetrated into the central part of the beam web. Some diagonal cracks were distributed in the joint panel and the flexural cracks were observed on column faces after a DR of 2.0%. By DR=3.0%, the crack pattern was denser in the beam plastic hinge region and the cracks widened. No more new joint diagonal cracks or column flexural cracks were observed until the end of the test. The cover concrete at the bottom beam chord at column faces showed incipient crushing after DR=3.5%. In

the load cycle to $DR=4.0\%$, damage on top and bottom beam chords adjacent to column faces spread further along beam span away from column faces and also toward the center of the beam height so that some part of longitudinal beam bars was exposed. Plastic hinge formation was clearly observed at the end of the test.

Furthermore, the energy dissipation of the specimen is shown in **Figure 5.4**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are decreasing after a DR of 3.5% for the first and second loading cycles. At $DR=4.0\%$, the energy dissipation value in the first loading cycle is 31% higher than that in the second cycle.

5.2.1.2 Beam behavior

The evaluation of strain distribution along beam reinforcement and the beam curvature values are discussed in the following sections. Strain profiles are shown for the positive and negative loading directions to $DR=1.0\%$ and $DR=3.0\%$ in order to represent the early and late loading stages.

Longitudinal beam bar strains

The strain profiles of the top beam reinforcement of EN80 are explained in **Figure 5.5**. As illustrated, the beam bar of EN80 started to yield in tension at the column face after the DR of 3.0% . This is compatible with the observed plastic hinges in beam ends. However, the strain values dropped dramatically in the joint center. This observation clarifies a good bond presence in the joint region.

Beam curvature distributions

Figure 5.6 shows the beam curvature distribution estimated from LVDT readings. Curvature values are presented for drift ratios of $DR=1.0\%$ and $DR=3.0\%$ in the positive and negative loading directions and yield curvatures are presented as dashed lines. It is observed that the beam curvatures measured near the column face reached the theoretical yield value after a DR of 1.0% . However, rapid increase of beam curvatures was observed in subsequent loading cycles as a result of the formation of plastic hinges in beam ends. However, it is observed that curvature values at a distance of 500 mm from the column face are smaller than yielding values.

Figure 5.1 Final crack pattern of Specimen EN80

Figure 5.2 Story shear force versus horizontal displacement for Specimen EN80

Figure 5.3 Progressive cracking development of Specimen EN80

Figure 5.4 Energy dissipation of Specimen EN80

* All dimensions are in *mm*

Figure 5.5 Strain profiles of beam bars in Specimen EN80

* All dimensions are in *mm*

Figure 5.6 Curvature distribution of beams of Specimen EN80

5.2.1.3 Column behavior

The evaluation of strains in the column side and center bars and column curvatures are discussed in the following sections. Strain profiles are shown for the positive and negative loading directions to DR=1.0% and DR=3.0% to represent the early and late loading stages.

Longitudinal column bar strains

Figure 5.7 shows the strain distribution along the column side and center bars. Some of the strain gauges were damaged during the concrete casting and the data

was not available. It is observed that strain values are far smaller than the yielding strain value in the side and center bars.

Column curvature distributions

Figure 5.8 depicts the column curvature distribution as per instrumented in the test. The curvature values measured at the upper and lower column were almost identical during the test. However, the curvature values at the beam face were larger than those at a distance of 300 mm from the beam face.

5.2.1.4 Joint behavior

The evaluation of joint behavior, bond stress of beam and column reinforcement, joint shear distortion and joint shear expansion is discussed in the sections that follow.

General behavior

In the loading to a DR of 1.0%, no crack was observed in the joint region. Initial diagonal tension and compression cracks occurred in the joint core in a loading to DR=1.5%. Maximum nominal horizontal shear stress of 4.0 MPa ($0.05 f'_c$) was observed in the joint core where f'_c is the measured compressive cylinder strength of concrete.

Bond stresses of the longitudinal beam and column bars in the joint

The average bond stresses along beam and column bars are plotted in **Figures 5.9** and **5.10**, respectively. The bond stress values were calculated at the peak of the selected loading cycles using the wire strain gauge readings. The average bond stress values along the longitudinal beam and column bars in the joint are assumed to be uniformly distributed over the gauge length. The maximum bond stress obtained along top beam bars was 2.3 MPa ($0.03 f'_c$). The bond stress is observed to increase at higher drift ratios, which is an indication of a steady bond condition. On the other hand, the column reinforcement remained elastic throughout the test with maximum bond stress of 1.5 MPa ($0.02 f'_c$) on the side bar and maximum bond stress of 2.6 MPa ($0.03 f'_c$) on the center bar, respectively.

Joint shear distortion and expansion

The measured joint shear distortion and expansion are shown in **Figure 5.11**. The estimation of joint shear distortion and expansion were described in **Chapter 3**.

* All dimensions are in *mm*

Figure 5.7 Strain profiles of column bars in Specimen EN80

* All dimensions are in *mm*

Figure 5.8 Curvature distribution of columns of Specimen EN80

* All dimensions are in *mm*

Figure 5.9 Bond stress of beam bars in Specimen EN80

* All dimensions are in *mm*

Figure 5.10 Bond stress of column bars in Specimen EN80

Figure 5.11 Joint shear distortion and expansion of Specimen EN80

It is observed the joint shear distortion and expansion had small values during the test and no significant change in the values was observed. This indicates that the joint remained elastic throughout the test. In the loading to DR=4.0%, the joint diagonal tension cracks opened wide with the maximum joint distortion of 0.1% and the maximum joint expansion of 0.1 mm.

5.2.1.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 5.12** for Specimen EN80. The major source of the story drift was the beam displacement, indicating a “strong-column and weak-beam” response. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 41% to 70% and 0.5% to 7%, respectively. The contribution of beam fixed-end rotation increased gradually. In addition, the contribution of column fixed-end rotation increased during the higher drift ratios whereas the contribution of column flexural deformation remained relatively low during the test. The contribution of column fixed-end rotation to the total drift was 4% to 8%. The maximum contribution of joint shear to the horizontal displacement was 13% at a DR of 2.0%. The contribution of joint shear distortion decreased slightly in subsequent loading cycles.

Figure 5.12 Contribution of components to horizontal displacement in EN80

5.2.2 Test Results of Specimen EH80

5.2.2.1 General behavior

The final cracking pattern of Specimen EH80 and the story shear force versus horizontal displacement relationship are shown in **Figures 5.13** and **5.14**. The specimen reached its theoretical capacity at a DR of 2.0% in the positive loading direction and did not reach the theoretical capacity in the negative loading direction. Maximum capacities of 51 kN in the positive and 91 kN in the negative loading directions were reached at the final loading cycles. These capacities were higher by 16% and 10% in comparison to those attained by EN80. The specimen managed to resist horizontal loading up to the drift ratio of 4.0% without any significant loss in lateral loading capacity.

The successive cracking pattern of the specimen is illustrated in **Figure 5.15**. In the load cycle to DR=1.0%, flexural cracks spread along the top and bottom chord of the beams. A couple of fine cracks appeared in the joint panel in the load run toward DR=1.0%. The first diagonal crack in the joint panel occurred in the negative load run toward DR=1.5% and it was located at the left upper corner of the joint panel. When the unit was taken to a displacement equal to DR=2.0%, diagonal cracks developed from corner to corner in the joint panel in the negative loading direction. No significant diagonal cracking was observed in the positive loading direction until the end of the test. More shear-flexural cracks were observed to spread along the beam spans at DR=2.0%. By DR=3.0%, the cracks in the plastic hinge region of beams widened further and the joint panel remained in the crack pattern without too much change. Compared to Specimen EN80, The number of flexural and shear-flexural cracks in the beam of EH80 is more, but the crack width and concrete damage is to a lesser degree. In addition, the number of cracks in the column and joint is higher in Specimen EH80, which indicates the effect of using HSS reinforcements. In the load cycle to DR=4.0%, cracking in the joint region remained essentially similar to that of previous loading cycles with crack widths remaining the same. The cover concrete around corners at the intersection of the beam and column began to crush, especially in the bottom corner.

Figure 5.13 Final crack pattern of Specimen EH80

Figure 5.14 Story shear force versus horizontal displacement for Specimen EH80

Figure 5.15 Progressive cracking development of Specimen EH80

Figure 5.16 Energy dissipation of Specimen EH80

Furthermore, the energy dissipation of the specimen is shown in **Figure 5.16**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are increasing continuously during the test for the first and second loading cycles. The energy dissipation value in the first loading cycle is 18% higher than that in the second cycle at the DR of 4.0%.

5.2.2.2 Beam behavior

The evaluation of strains in beam reinforcing bars and the beam curvatures are discussed in the following sections. Strain profiles are shown for the positive and negative loading directions to DR=1.0% and DR=3.0%.

Longitudinal beam bar strains

Figure 5.17 shows the strain profiles of the top and bottom beam bars of EH80. As illustrated, the strain values in beam bars did not reach the yielding value until DR=3.0%. In general, strain values along the bars in the beam section and the joint panel did not experience any significant change, unless in the center of the bottom bar inside the joint. This observation could be due to the strain gauge damage and false readings. Furthermore, bond deterioration in the joint region is observed to be more severe in this specimen with higher-grade reinforcements.

Beam curvature distributions

Figure 5.18 shows the beam curvature distributions estimated from transducer readings. Curvature values are presented for drift ratios of DR=1.0% and DR=3.0% in the positive and negative loading directions. It is observed that the beam curvatures near the column face reached the theoretical yield value after a DR of 1.0%. However, the beam curvatures increase was rapid in subsequent loading cycles in both of the loading directions mainly as a result of the formation of plastic hinges in the beam ends.

* All dimensions are in *mm*

Figure 5.17 Strain profiles of beam bars in Specimen EH80

* All dimensions are in *mm*

Figure 5.18 Curvature distribution of beams of Specimen EH80

5.2.2.3 Column behavior

The evaluation of strains in column side reinforcement, column middle reinforcement and column curvatures are discussed in the following sections. Strain profiles are shown for the positive and negative loading directions to DRs of 1.0% and 3.0%.

Longitudinal column bar strains

The strains of the column side and center bars are shown in **Figure 5.19**. It is observed that during positive and negative loading cycles, the side and center bars experience both tension and compression. However, the value of compression strains is much smaller than tension strains. The strain values in higher drift ratios are slightly increasing. Although the strain values are still smaller than the yielding value, strain values are higher than those in Specimen EN80 with normal-strength steel reinforcements.

Column curvature distributions

Figure 5.20 shows the column curvature distribution as per instrumented in the test. During loading cycles to DR=1.0%, the curvature values at the top and bottom columns were almost similar and they were smaller than the yielding curvature value. At higher drift ratios, the curvature values at the negative loading direction were slightly larger than those during positive loadings.

* All dimensions are in *mm*

Figure 5.19 Strain profiles of column bars in Specimen EH80

* All dimensions are in *mm*

Figure 5.20 Curvature distribution of column of Specimen EH80

5.2.2.4 Joint behavior

The bond stress of beam and column reinforcement, joint shear distortion and joint shear expansion are evaluated in the following sections to clarify the joint behavior.

General behavior

A maximum nominal horizontal shear stress of 4.4 MPa ($0.06 f'_c$) was observed in the joint core. No diagonal crack was observed in the joint region up to a loading to a DR of 1.0%. Initial diagonal tension and compression cracks occurred in the joint core in the first loading cycle to DR=1.5%.

Bond stresses of the longitudinal beam and column bars in the joint

Figures 5.21 and 5.22 show the average bond stresses for beam and column bars, respectively. The average bond stresses measured along the longitudinal beam and column bars in the joint are assumed to be uniformly distributed over the gauge length. The maximum obtained bond stress along the top beam bars was 1.5 MPa ($0.02 f'_c$), while the maximum bond stress along the bottom beam bars was 5.4 MPa ($0.07 f'_c$). The bond stress along the beam bars is observed to increase at higher drift ratios, which is an indication of a steady bond condition. The longitudinal column reinforcements remained elastic until the end of the test with maximum bond stress of 2.2 MPa ($0.03 f'_c$) on the side and 1.7 MPa ($0.02 f'_c$) on the center bars.

Joint shear distortion and expansion

Figure 5.23 illustrates the measured joint shear distortion and expansion. It is observed the joint shear distortion and expansion had small values at DR=1.0%, while they experience a significant increase in following drift ratios to the end of the test. In the loading to DR=4.0%, the joint diagonal tension cracks opened wide with the maximum joint distortion of 0.4% and the maximum joint expansion of 1.0 mm.

* All dimensions are in *mm*

Figure 5.21 Bond stress of beam bars in Specimen EH80

* All dimensions are in *mm*

Figure 5.22 Bond stress of column bars in Specimen EH80

Figure 5.23 Joint shear distortion and expansion of Specimen EH80

5.2.2.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 5.24** for Specimen EH80. The major source of the story drift was the beam displacement. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 18% to 48% and 12% to 18%, respectively. The contribution of beam fixed-end rotation increased gradually, while the contribution of beam flexural rotation decreased significantly at the final loading cycle. In addition, the contribution of column fixed-end and flexural rotation did not change significantly during the test. The contribution of column fixed-end and column flexural rotation to the total drift was 4% to 5% and 0.7% to 1%. The maximum contribution of joint shear to the horizontal displacement was 13% at a DR of 2.0%. The contribution of joint shear distortion decreased slightly in subsequent loading cycles.

Figure 5.24 Contribution of components to horizontal displacement in EH80

5.2.3 Test Results of Specimen EH80A

5.2.3.1 General behavior

The final cracking pattern of Specimen EH80A and the horizontal story shear force versus horizontal displacement hysteresis loops are shown in **Figures 5.25** and **5.26**. The specimen reached its theoretical capacity at a DR of 1.5% in positive and at a DR of 2.0% in the negative loading direction. The maximum attained capacities were 53 kN in the positive and 120 kN in the negative loading directions.

The progressive cracking pattern of the specimen is shown in **Figure 5.27**. In the load cycle corresponding to DR=1.0%, Flexural cracks occurred in the top and bottom beam chords during the positive and negative loading directions and no shear cracks were observed. Similar to the other specimens with column axial loading, no cracks were observed in columns and in the joint region. In the load cycle to DR=2.0%, more flexural cracks occurred in beam plastic hinge regions. Moreover, new diagonal cracks were observed in beams, while some existing cracks changed direction and became diagonally oriented. By DR=3.0%, flexural cracks in beam plastic hinge regions continue to widen and new cracks occurred in this region. As the test proceeded toward DR=4.0%, beam bottom chords adjacent to the column were crushed significantly and large gap openings occurred in beam plastic hinge regions. Beam bottom bars were exposed at the end of the test.

Furthermore, the energy dissipation of the specimen is shown in **Figure 5.28**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are increasing continuously during the test for the first and second loading cycles. At DR=4.0%, the energy dissipation value in the first loading cycle is 24% higher than that in the second cycle.

5.2.3.2 Beam behavior

The beam behavior is discussed via the evaluation of strains in beam reinforcement and beam curvatures as follows:

Figure 5.25 Final crack pattern of Specimen EH80A

Figure 5.26 Story shear force versus horizontal displacement for Specimen EH80A

(a) Drift Ratio of 1.0%

(b) Drift Ratio of 2.0%

(a) Drift Ratio of 3.0%

(b) Drift Ratio of 4.0%

Figure 5.27 Progressive cracking development of Specimen EH80A

Figure 5.28 Energy dissipation of Specimen EH80A

Longitudinal beam bar strains

Figure 5.29 explains the strain profiles of the top and bottom beam reinforcement of Specimen EH80A. It is observed that up to the DR of 1.0%, the beam reinforcement was in elastic range as none of the strain gauges exceeded its yield strain limit. Plastic hinge was formed at the beam section adjacent to the column face when DR of 4.0% was attained. Strain values in the joint region were not increasing during the test, which is because of the effect of column axial loading.

Beam curvature distributions

Figure 5.30 shows the beam curvature distributions estimated from transducer readings. The beam curvature measured near the column face almost reached the theoretical yield curvature in the loading stage of DR=1.0%. Beam curvature increases rapidly after the DR of 3.0% and exceeded its theoretical yield curvature. This observation is compatible with test observations.

5.2.3.3 Column behavior

The column behavior is evaluated based on the measured strains in column side reinforcement and column middle reinforcement. The column curvature was obtained from the measured displacement along the column as described in **Chapter 3**.

Longitudinal column bar strains

The strains of the column side and center bars are explained in **Figure 5.31**. Due to the confinement effect of the column axial compression loading, the column basically remained elastic throughout the test. Although the strain of column bars increase gradually, they did not reach the yield strain. The strain values are much smaller than those in EH80, as a result of the presence of column axial loading.

Column curvature distributions

Figure 5.32 shows the column curvature distributions as per instrumented in the test. As the column was still in its elastic range, the respective curvatures increased gradually but no rapid increase was observed.

* All dimensions are in *mm*

Figure 5.29 Strain profiles of beam bars in Specimen EH80A

* All dimensions are in *mm*

Figure 5.30 Curvature distribution of beams of Specimen EH80A

* All dimensions are in *mm*

Figure 5.31 Strain profiles of column bars in Specimen EH80A

* All dimensions are in *mm*

Figure 5.32 Curvature distribution of columns of Specimen EH80A

5.2.3.4 Joint behavior

The bond stress of beam and column reinforcing bars, joint shear distortion and joint shear expansion are discussed in the sections that follow.

General behavior

For EH80A, the maximum nominal horizontal shear stress in the joint core was 4.2 MPa ($0.05 f'_c$). As shown in **Figure 5.27**, no diagonal cracking was observed in the joint region because of the column axial loading.

Bond stresses of the longitudinal beam and column bars in the joint

Figures 5.33 and **5.34** show the average bond along the beam and column bars. The maximum bond stress obtained along top beam bars was 1.2 MPa ($0.01 f'_c$), while the maximum bond stress observed along the bottom beam bars was 0.6 MPa ($0.007 f'_c$). On the other hand, the column reinforcement remained elastic until the end of the test with maximum bond stress of 0.6 MPa ($0.007 f'_c$) on the side bar.

Joint shear distortion and expansion

Figure 5.35 explains the measured joint shear distortion and expansion of EH80A. As observed, the joint shear distortion and expansion are generally small as a result of the confinement effects of column axial compression loading. In the final loading to DR=3.0%, the maximum joint distortion is 0.01% and the maximum joint expansion is 0.06 mm.

* All dimensions are in *mm*

Figure 5.33 Bond stress of beam bars in Specimen EH80A

* All dimensions are in mm

Figure 5.34 Bond stress of column bars in Specimen EH80A

Figure 5.35 Joint shear distortion and expansion of Specimen EH80A

5.2.3.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 5.36** for Specimen EH80A. The major source of the story drift was the beam displacement. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 23% to 53% and 2% to 10%, respectively. The contribution of beam fixed-end rotation increased gradually. In addition, the contribution of column fixed-end rotation did not change during the test. Due to the presence of column axial loading, the contribution of column flexural rotation was negligible. The contribution of column fixed-end rotation to the total drift was 0.9% to 3%. Furthermore, the contribution of joint shear distortion had a small value during the test and it was controlled by the presence of column axial loading.

Figure 5.36 Contribution of components to horizontal displacement in EH80A

5.3 Test Result of Group Five (EH60 and EH60A)

Group Five consists of specimens EH60 and EH60A with same dimension as described in **Chapter 3**. Both of the specimens were HSC beam-column joints with HSS reinforcement. EH60 was tested without column axial loading while Specimen EH60A was tested with column axial compression loading of $0.3 f'_c A_g$.

The typical features of these specimens are summarized as follows:

- Smallest diameter of top and bottom bars in the beam section = 19 mm
- Actual development length of longitudinal beam bars (d_b/l_{dh}) = 1/21
- Column-bar to beam-depth ratio = 1/23
- Calculated development length of longitudinal beam bars (d_b/l_{dh}) based on ACI318-11 = 1/17
- Calculated maximum column-bar-to-beam-depth based on clause 10.4.6.6 in NZS 3101-06 [N3] = 1/28
- Joint core reinforcement detailing complied with the requirement in clause 15.4.4.4 in NZS 3101-06 [N3] (maximum bar spacing = 85 mm, allowable bar spacing = 160 mm)

The following sections report the test result of Specimens EH60 and EH60A.

5.3.1 Test Results of Specimen EH60

5.3.1.1 General behavior

Figures 5.37 and **5.38** show the final cracking patterns of Specimen EH60 and the measured horizontal story shear force versus horizontal displacement hysteresis loops. The specimen reached its theoretical capacity at DRs of 2.5% and 2.0% in the positive and negative loading direction, respectively. The maximum attained capacities were 69 kN in the positive and 86 kN in the negative loading directions. Up to the final loading cycle, the residual load-carrying capacity was 64 kN in the positive and 85 kN in the negative loading direction. These values were larger than the 80% of the maximum attained force.

The cracking pattern of Specimen EH60 in various drift ratios is shown in **Figure 5.39**. In the load cycles to DR=1.0%, flexural cracks in the positive and negative loading directions propagated in a vertically upward or downward direction

covering full depth of the beam and spreading along half of the beam span. Some shear cracks were observed on the beam portion. Several diagonal cracks developed in the joint region at the first cycle to $DR=1.5\%$ while a few flexural cracks were developed on column faces. By $DR=2.0\%$, shear-flexural cracks appeared along the beam span and more diagonal cracks occurred in the joint region in the both loading directions. In loading runs to $DR=3.0\%$, some more fine cracks developed in the joint panel. In addition, the existing flexural cracks changed the direction and continued to spread in a 45 degree direction. At this stage, the flexural cracks in the beam plastic hinge region began to widen. At $DR=4.0\%$, incipient crushing of the cover concrete on top and bottom chord of the beam at the column face was observed. As the test proceeded toward the second loading cycle to $DR=4.0\%$, the cracking pattern in beams and in the joint region was similar to that of previous loading cycles. Concrete crushing was in progress in the two corners and the split of cover concrete was observed along the top and bottom beam bars. Crushing of concrete on the top and bottom beam chords adjacent to column faces spread along the column face vertically and covered the whole beam section.

Furthermore, the energy dissipation capacity of the specimen is shown in **Figure 5.40**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are increasing continuously during the test for the first and second loading cycles. At $DR=4.0\%$, the energy dissipation value in the first loading cycle is 5% higher than that in the second cycle.

5.3.1.2 Beam behavior

The strain profiles of beam reinforcement as well as the beam curvature distribution are explained in the sections that follow.

Longitudinal beam bar strains

As illustrated in **Figure 5.41**, the bottom beam reinforcements of Specimen EH60 showed a sign of yielding during positive loading cycles at the column face. However, large strain values were observed in the joint center.

Figure 5.37 Final crack pattern of Specimen EH60

Figure 5.38 Story shear force versus horizontal displacement for Specimen EH60

(a) Drift Ratio of 1.0%

(b) Drift Ratio of 2.0%

(a) Drift Ratio of 3.0%

(b) Drift Ratio of 4.0%

Figure 5.39 Progressive cracking development of Specimen EH60

Figure 5.40 Energy dissipation of Specimen EH60

* All dimensions are in *mm*

Figure 5.41 Strain profiles of beam bars in Specimen EH60

Beam curvature distributions

The beam curvature distributions which were estimated based on transducer readings are depicted in **Figure 5.42**. Due to the plastic hinge formation in the beam end, beam curvature values near the column face exceeded the theoretical yield curvature at the DR of 1.0%. A sharp increase in curvature values was observed in subsequent loading cycles, which shows the high energy dissipation of beams.

5.3.1.3 Column behavior

The evaluation of strains in column reinforcement besides the column curvature distributions are discussed in the sections that follow.

Longitudinal column bar strains

Generally, the strain values along the column bars did not reach the yield strain as explained in **Figure 5.43**. The column side bars experienced a variation between the compression and tension strain, while the center bars experienced only the tension strain during the test.

Column curvature distributions

The column curvature distribution as per instrumented in the test is explained in **Figure 5.44**. Curvature values were smaller than yield curvature during the DR of 1.0%. Although it was evident that the column remained elastic during the test, an increase in curvature values is observed in DR=3.0%.

* All dimensions are in *mm*

Figure 5.42 Curvature distribution of beams of Specimen EH60

Figure 5.43 Strain profiles of column bars in Specimen EH60

* All dimensions are in *mm*

Figure 5.44 Curvature distribution of columns of Specimen EH60

5.3.1.4 Joint behavior

The evaluation of joint behavior on bond stress of beam and column reinforcement, joint shear distortion and joint shear expansion are discussed in the sections that follow.

General behavior

The diagonal tension cracks were initially observed at a DR of 1.5% as explained in **Figure 5.39**. The cracks were then propagated in the subsequent loading cycles. The maximum nominal horizontal shear stress in the joint core was 3.3 MPa ($0.06 f'_c$).

Bond stresses of the longitudinal beam and column bars in the joint

The average bond stress measured along the longitudinal beam and column bars in the joint was calculated using the wire strain gauge readings. These calculated bond stresses are plotted in **Figure 5.45** for the beam bars and **Figure 5.46** for the column bars.

The maximum bond stress along top beam bars was 1.7 MPa ($0.03 f'_c$), while the maximum bond stress along the bottom beam bars was 2.6 MPa ($0.04 f'_c$). The bond behavior of the specimen was satisfactory throughout the test as the bond values increased in larger drift ratios. Furthermore, the maximum bond stress of 1.6 MPa ($0.03 f'_c$) was obtained along the column side bars, while the value of 1.9 MPa ($0.03 f'_c$) was obtained along the column center bars.

Joint shear distortion and expansion

Figure 5.47 illustrates the measured joint shear distortion and expansion. It is observed the joint shear distortion and expansion had small values until DR=2.0%, while they experienced a noticeable increase up to a DR of 4.0%. In the loading to DR=4.0%, the joint diagonal tension cracks opened wide with the maximum joint distortion of 0.2% and the maximum joint expansion of 0.5 mm.

* All dimensions are in *mm*

Figure 5.45 Bond stress of beam bars in Specimen EH60

* All dimensions are in *mm*

Figure 5.46 Bond stress of column bars in Specimen EH60

Figure 5.47 Joint shear distortion and expansion of Specimen EH60

5.3.1.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 5.48** for Specimen EH60. The major source of the story drift was the beam displacement. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 12% to 63% and 10% to 12%, respectively. The contribution of beam fixed-end rotation increased gradually, while the contribution of beam flexural rotation did not change significantly during the test. In addition, the contribution of column fixed-end rotation increased slightly in higher drift ratios. However, the contribution of column flexural deformation was relatively small during the test. The contribution of column fixed-end rotation to the total drift was 6% to 12%. The maximum contribution of joint shear to the horizontal displacement was 3% at a DR of 4.0%. The contribution of joint shear distortion increased gradually in higher drift ratios.

Figure 5.48 Contribution of components to horizontal displacement in EH60

5.3.2 Test Results of Specimen EH60A

5.3.2.1 General behavior

The final cracking pattern of Specimen EH60A and the horizontal story shear force versus horizontal displacement hysteresis loops are shown in **Figures 5.49** and **5.50**. This specimen reached its theoretical capacity at the DR of 4.0% and 2.0% during the positive and negative loading directions, respectively. The maximum attained capacities were 69 – in the positive and 83 kN in the negative loading directions. These capacities were similar to those attained by Specimen EH60. Up to the final loading cycle, the residual load-carrying capacity was increasing and no decay in the strength was observed.

The progressive cracking pattern for Specimen EH60A is shown in **Figure 5.51**. In the load cycle to DR=1.0%, flexural cracks spread along the top and bottom chord of the beams and no cracks were observed in columns and in the joint panel. By DR=2.0%, some of the beam flexural cracks changed their direction and shear-flexural cracks were formed. At DR=3.0%, plastic hinges developed in the beams with more flexural cracks forming and widening of existing cracks. In the loading run to DR=4.0%, the cracking pattern in the beam remained unchanged, but the cover concrete in the beam plastic hinge region began to spall off at this stage.

Moreover, the width of the vertical crack in the beam plastic hinge was increasing continuously until the end of the test. No significant cracking was observed in the column and the joint region.

Furthermore, the energy dissipation of the specimen is shown in **Figure 5.52**. The data is shown for the first and second cycles of the loading procedure. It is observed that the values are increasing continuously during the test for the first and second loading cycles. At DR=4.0%, the energy dissipation value in the first loading cycle is 13% higher than that in the second cycle.

5.3.2.2 Beam behavior

The following sections discuss the evaluation of strains in beam reinforcement in addition to beam curvature distributions:

Figure 5.49 Final crack pattern of Specimen EH60A

Figure 5.50 Story shear force versus horizontal displacement for Specimen EH60A

Figure 5.51 Progressive cracking development of Specimen EH60A

Figure 5.52 Energy dissipation of Specimen EH60A

Longitudinal beam bar strains

As explained in **Figure 5.53**, there is no yielding along the top and bottom beam reinforcements at DR=1.0%. However, the reinforcements near the column face experienced high strain in the lateral loading to DR=3.0%. The strain gauges at the tension face reached the yielding value while the strain at the compression face was relatively low.

Beam curvature distributions

Figure 5.54 shows the estimated beam curvature distributions based on transducer readings. In the early loading stage to DR=1.0%, the beam curvature value near the column face almost reached the theoretical yield curvature. In subsequent loading cycles, the curvature values surpassed its yield curvature limit. A similar trend was also observed in negative loading cycles because of the plastic hinge formation in the beam end.

5.3.2.3 Column behavior

The evaluation of strains in the column side and center bars as well as the column curvatures are discussed in the sections that follow:

Longitudinal column bar strains

As shown in **Figure 5.55**, the column basically remained elastic throughout the test because of the confinement effect of column axial compression loading. Column bars did not reach yielding strain values and strain values in this specimen are much smaller than those in Specimen EH60.

Column curvature distributions

Figure 5.56 shows the column curvature distributions. The column was in its elastic range at a DR of 1.0% and the respective curvatures increased gradually until the late stage loading of DR=3.0%.

* All dimensions are in *mm*

Figure 5.53 Strain profiles of beam bars in Specimen EH60A

* All dimensions are in *mm*

Figure 5.54 Curvature distribution of beams of Specimen EH60A

* All dimensions are in *mm*

Figure 5.55 Strain profiles of column bars in Specimen EH60A

* All dimensions are in *mm*

Figure 5.56 Curvature distribution of columns of Specimen EH60A

5.3.2.4 Joint behavior

The joint behavior discussion shall cover the bond stress of beam and column reinforcement, joint shear distortion and joint shear expansion.

General behavior

Due to the confinement effect of the applied column axial compression loading, noticeable cracks were not observed in the joint core and the column section of EH60A throughout the test. The maximum nominal horizontal shear stress of 2.9 MPa ($0.05 f'_c$) was observed.

Bond stresses of the longitudinal beam and column bars in the joint

The average bond stress along the beam and column reinforcing bars are shown in **Figures 5.57** and **5.58**, respectively. The maximum bond stress of bottom beam bars was 2.9 MPa ($0.04 f'_c$). However, bond deterioration was observed as strain values decreased in higher drift ratios. The maximum bond stress of 0.7 MPa ($0.01 f'_c$) and 0.9 MPa ($0.01 f'_c$) was observed on the column side and center bars, respectively.

Joint shear distortion and expansion

The measured joint shear distortion and expansion are shown in **Figure 5.59**. It was observed that the joint shear distortion and expansion values remained relatively small until a DR of 3.0%. Due to the column axial loading, the joint panel remained elastic during the test and negligible joint deformation was observed. However, a slight increase in values is observed at the final loading cycle and a final joint shear expansion of 0.4 mm and the joint shear distortion of 0.07% were observed at the end of the test.

* All dimensions are in *mm*

Figure 5.57 Bond stress of beam bars in Specimen EH60A

* All dimensions are in *mm*

Figure 5.58 Bond stress of column bars in Specimen EH60A

Figure 5.59 Joint shear distortion and expansion of Specimen EH60A

5.3.2.5 Decomposition of horizontal displacement

The contribution of each component to the horizontal displacement is shown in **Figure 5.60** for Specimen EH60A. The major source of the story drift was the beam displacement. The contributions of beam fixed-end and flexure rotation to the total drift were ranging from 24% to 60% and 7% to 17%, respectively. The contribution of beam fixed-end rotation increased gradually, while the contribution of beam flexural rotation decreased during the loading cycles. In addition, the contribution of column fixed-end rotation was also increasing slightly during the test. As a result of the presence of column axial loading, the contribution of column flexural rotation to the horizontal displacement was negligible. The contribution of column fixed-end rotation to the total drift was 3% to 10%. Furthermore, the contribution of joint shear was around 1% because of the column axial loading.

Figure 5.60 Contribution of components to horizontal displacement in EH60A

5.4 Conclusions

The test result of five exterior beam-column joints constructed with high-strength materials was presented. The specimens were divided into two groups based on their reinforcement detailing. Based on test results and observation, the following conclusions are reached:

1. The performance of HSC exterior specimens including the lateral loading capacity and energy dissipation capacity found to be satisfactory until a DR of 4.0%.
2. All of the specimens met the bond development requirement specified by NZS3101-06 [N3]. The energy dissipation capacity of all the specimens was acceptable according to ACI T1.1-02 [A2], which shows the applicability of NZS3101-06 [N3] provisions in case of using high-strength materials.
3. The strain profile of reinforcements showed that the failure mode of the specimens is controlled by the formation of plastic hinges in the beam end facing the column. This observation is in agreement with the strong-column and weak-beam philosophy.
4. Similar to interior joints, flexural cracks occurred in beams and columns of the exterior joints during the test. However, the cracks in the column section did not widen through the test. In addition, limited diagonal tension cracking was observed in the joint panel.
5. Generally, the presence of axial compression loading enhanced the energy dissipation capacity of the specimens. However, no correlation was observed between the axial column loading and the maximum load attained by the specimens.
6. The required development length of longitudinal beam bars (l_{dh}/d_b) is increased significantly when the reinforcement with higher yield-strength is used. Utilizing HSC coupled with HSS reinforcement and also the incorporation of the effect of column axial loading are the key solutions to overcome this issue.

CHAPTER 6

DISCUSSION ON TEST RESULTS OF BEAM-COLUMN JOINTS

6.1 Introduction

This chapter presents the discussion on experimental results of the seven interior and five exterior joints testes in this study. The discussion covers the comparison of beam curvatures, secant stiffness, energy dissipation capacity, viscous damping, decomposition of lateral displacement, joint shear strength, joint constitutive modeling, bond behavior, and the influence of varying axial compression loading.

6.2 Comparison of Beam Curvatures of Test Specimens

As discussed in **Chapters 4** and **5**, it was observed that the beam curvatures measured near the column face for all of the specimens reached the theoretical yield value after a DR of 1.0%. However, rapid increase was observed in beam curvatures in the subsequent loading cycles as a result of the formation of plastic hinges in beam ends. It is observed that beam curvature values are relatively smaller in Specimens IH80 and IH100 with HSS reinforcements compared to Specimens IN80 and IN 100. In addition, Specimen IH60 with T19 beam bars also showed smaller curvatures compared to other specimens with T16 beam bars. However, the role of concrete strength and column axial load was insignificant on beam curvatures.

6.3 Comparison of Secant Stiffness of Test Specimens

The experimental secant stiffness was estimated according to the hysteresis response of the specimens. **Figure 6.1** shows the stiffness of the specimens versus the story drift ratios. It is observed that using high- strength reinforcements and also the level of axial compression load play a significant role on the stiffness of the specimens, whereas the influence of concrete grade proved to be negligible. In all story drift ratios, the secant stiffness of the specimens with high-strength steel (HSS) reinforcements was higher than that of the identical specimens with ordinary reinforcements.

Figure 6.1 Secant stiffness degradation of the test specimens

The specimens with HSS reinforcements showed lower stiffness degradation such that their stiffness was around 50% higher than that of the specimens with normal-strength reinforcements at a DR of 4.0%. In addition, the axial compression loading had a notable influence on increasing the stiffness degradation of the specimens. The stiffness of Specimen IH80A was 10% higher than that of Specimen IH80 at a DR of 1.0%. However, both of the specimens had the same stiffness at a DR of 2.5%. At the final loading cycle, the stiffness of Specimen IH80A was only 78% of the stiffness of Specimen IH80. The same trend was observed in Specimens IH60 and IH60A. The secant stiffness of Specimen IH60A was 40% higher than that of Specimen IH60 during the initial loading cycles, while the specimens showed the same stiffness at a DR of 2.5%. The both specimens showed almost equal stiffness degradation after DR of 2.5% until the end of the test.

6.4 Energy Dissipation Capacity of Test Specimens

Energy dissipation capacities were calculated based on the enclosed area of each loop in hysteresis response of the specimens and the result is illustrated in **Figure 6.2**. It is observed that the presence of axial loads increased the energy dissipation significantly and utilizing HSS reinforcements resulted in a smaller energy dissipation capacity. However, Specimen IH60 with a larger beam bar diameter of T19 showed a smaller energy dissipation compared to others.

The performance requirement of “Acceptance Criteria for Moment Frames Based on Structural Testing (ACI T1.1-01)” [A2] is presented in **Figure 6.3**. A method to calculate the normalized energy dissipation and the recommended minimum values for the energy dissipation capacity is proposed by ACI T1.1-01 [A2]. In order to be compared with the performance requirements of ACI T1.1-01 [A2], the normalized energy dissipation capacity of the specimens were calculated and are shown in **Figure 6.4**. The minimum permissible value for the dissipated energy is presented as the dashed line. The general trend indicates that in lower story drift ratios, when limited cracking has happened and the behavior of the specimens is almost ideal elasto-plastic, the energy dissipation value is higher than that at the end of the test. However, it is observed that the energy dissipation capacity of all the specimens is above the recommended limit.

Figure 6.2 Energy dissipation capacity of specimens

Figure 6.3 Normalization of energy dissipation capacity following ACI T1.1-01

[A2]

Figure 6.4 Normalized energy dissipation capacity for second cycles of specimens

6.5 Viscous Damping of Test Specimens

In order to further quantify the measurement of the dissipated energy in each loading cycle, the equivalent viscous damping ξ_{eq} was computed for the specimens as shown in **Figure 6.5**. Generally, all the specimens showed similar damping properties. At initial loading stages, Specimens IN100 and IH80A showed considerably higher viscous damping compared to other specimens. However, this high damping value decreased rapidly and the damping value for all of the specimens became almost the same at a DR of 1.0%. As observed, the equivalent viscous damping of the specimens was generally decreasing up to a story drift ratio of 1.5% and after this point, the viscous damping increased as drift ratio increased. At the end of the test, Specimens IN100 and IH80A showed higher viscous damping compared to the others yet. As a general observation, the specimens with normal-strength reinforcements showed to have a 75% higher viscous damping value compared to the identical specimens with HSS reinforcements at the end of the test. The higher value of the equivalent viscous damping is an indication of a less pinching hysteretic behavior of the specimens.

6.6 Decomposition of Lateral Displacement of Test Specimens

Figure 6.6 shows the decomposition percentages of the lateral displacement measured at the top of the column at a DR of 4.0%. It is observed that the highest contribution to the lateral displacement belongs to the beam fixed end and flexural rotations, indicating a strong column-weak beam response. However, it is observed that the contribution of beam rotations is smaller in specimens with higher grade of concrete or steel reinforcements. Furthermore, it can be seen that the joint shear distortion is not a significant source of lateral displacements because of its high rigidity. In specimens with axial loadings, the contribution of beam rotations is increased and the role of the joint shear distortion and column flexural rotations can be neglected. These findings are totally compatible with the observed cracking pattern and failure modes of the testing specimens.

Figure 6.5 Equivalent viscous damping of specimens

Figure 6.6 Decomposition of lateral displacement of specimens at DR=4%

6.7 Evaluation of Joint Shear Strength

The joint shear strength γ of all specimens was compared with ASCE/SEI 41-06 [A7] and ACI 352-02 [A3] recommendations as presented in **Table 6.1**. As shown in **Figure 6.7**, the test results proved that ASCE/SEI 41-06 [A7] and ACI 352-02 [A3] recommendations were unconservative in predicting the joint shear performance of HSC specimens with HSS reinforcements. The γ values of the specimens with T16 HSS bars were between 44 and 48% of the value suggested by ASCE/SEI 41-06 and ACI-ASCE 352-02. However, this ratio was around 65% for the specimens with T19 HSS bars and between 65 to 68% for the specimens with an axial load level of $0.3 f'_c A_g$. Moreover, it is observed that in specimens with a high axial load level, the value of joint shear stress is moderately higher. The reason for this observation is that any increase in the column compression will extend the compression zone further along the joint boundaries. As a result, inclination of the diagonal compression strut will decrease and its contribution in resisting the shear force will become less remarkable. Consequently, it is observed that IH60A and IH80A attract larger shear forces in their joint region.

The acceptability of force and deformation actions in joint members is evaluated using the model presented in ASCE/SEI 41-06. This model is shown in **Figure 6.8**. The experimentally obtained values of parameter “d” for the specimens tested in this study are presented in **Table 6.1**. It is observed that the values suggested by ASCE/SEI 41-06 are overestimating the shear ductility of the joints in all test specimens. The comparison between the specimens reinforced with HSS bars and ordinary reinforcements clearly shows that the value of “d” is decreased slightly when HSS reinforcements are utilized. Furthermore, it is observed that the axial loading has a crucial effect on the shear behavior of joints as shear strain values are significantly smaller in Specimens IH80A and IH60A.

Figure 6.7 Joint shear stress of specimens

Table 6.1 Experimental values of joint shear strains and stresses in comparison with ASCE/SEI 41-06

Specimen	f'_c (MPa)	Joint shear stress(experimental)			ASCE/SEI41			
		v_{jh} (MPa)	v_{jh} / f'_c (MPa)	γ (MPa)	γ (MPa)	γ / γ_{ASCE41} (%)	$d_{(Test)}$ (rad)	$d_{(ASCE41)}$ (rad)
IN80	80	6.0	$0.08 \sqrt{f'_c}$	$0.68 \sqrt{f'_c}$	$1.245 \sqrt{f'_c}$	54.2	0.003	0.015
IH80	80	5.4	$0.07 \sqrt{f'_c}$	$0.60 \sqrt{f'_c}$	$1.245 \sqrt{f'_c}$	48.2	0.001	0.015
IH80A	80	7.6	$0.10 \sqrt{f'_c}$	$0.85 \sqrt{f'_c}$	$1.245 \sqrt{f'_c}$	68.6	0.000	0.015
IN100	100	4.1	$0.04 \sqrt{f'_c}$	$0.41 \sqrt{f'_c}$	$1.245 \sqrt{f'_c}$	33.2	0.002	0.015
IH100	100	5.5	$0.06 \sqrt{f'_c}$	$0.55 \sqrt{f'_c}$	$1.245 \sqrt{f'_c}$	44.2	0.002	0.015
IH60	60	6.2	$0.10 \sqrt{f'_c}$	$0.80 \sqrt{f'_c}$	$1.245 \sqrt{f'_c}$	64.6	0.002	0.015
IH60A	60	6.3	$0.10 \sqrt{f'_c}$	$0.81 \sqrt{f'_c}$	$1.245 \sqrt{f'_c}$	65.3	0.000	0.015
EN80	80	4.03	$0.05 \sqrt{f'_c}$	$0.45 \sqrt{f'_c}$	$0.996 \sqrt{f'_c}$	45.1	0.003	0.01
EH80	80	4.43	$0.06 \sqrt{f'_c}$	$0.50 \sqrt{f'_c}$	$0.996 \sqrt{f'_c}$	50.2	0.001	0.01
EH80A	80	4.22	$0.05 \sqrt{f'_c}$	$0.47 \sqrt{f'_c}$	$0.996 \sqrt{f'_c}$	47.2	0.000	0.01
EH60	60	3.35	$0.06 \sqrt{f'_c}$	$0.43 \sqrt{f'_c}$	$0.996 \sqrt{f'_c}$	43.2	0.002	0.01
EH60A	60	2.87	$0.05 \sqrt{f'_c}$	$0.37 \sqrt{f'_c}$	$0.996 \sqrt{f'_c}$	37.1	0.000	0.01

Figure 6.8 Joint shear stress versus shear angle by ASCE/SEI 41-06

6.8 Joint Constitutive Modeling

In order to implement the constitutive relationship of the joints in the finite-element modeling, the backbone curve of the joint response should be determined first. In the current study, a joint model is developed based on the joint shear stress-strain relationship of the experimentally tested high-strength beam-column joints. Based on test observations, it is concluded that the constitutive model with a multi-linear envelope as shown in **Figure 6.9** can represent the shear behavior of specimens. The envelope consists of five lines starting from the origin, which are connecting four key points and these control points are to be defined. Joint shear parameters of seven HSC beam-column joints tested in the current study are summarized in **Table 6.2**. It can be suggested that the four key points of the backbone curve for the shear response of joints correspond to the joint shear cracking, the yielding capacity of beams, the ultimate capacity of beams and the residual strength. Based on the experimental results presented in **Table 6.2**, joint shear stress corresponding to the joint shear cracking is observed to be within the range of $0.1-0.3\sqrt{f'_c}$. The proposed **Equation 6.1** predicts the cracking joint shear stress well for beam-column joints reinforced with steel reinforcements of yielding capacities higher than 550 MPa:

$$(\tau_{jh})_{cr} = 1.7 \sqrt{1 + 0.01 \left(\frac{P}{A_{jh}} \right)} \quad (MPa) \quad (6.1)$$

Where P is the axial compression load and A_{jh} is the joint area. However, the values of $(\tau_{jh})_y$ and $(\tau_{jh})_{\max}$ are determined based on the section analysis carried out to find the yield and ultimate capacities of beams and columns. It was observed that the values of $(\tau_{jh})_y$ and $(\tau_{jh})_{\max}$ fall within the range of $0.2-0.6\sqrt{f'_c}$ and $0.5-0.7\sqrt{f'_c}$. The values of joint shear strains are required in order to determine the ordinates of the backbone key points. The experimental data shows that the shear strains $(\gamma_j)_{cr}$, $(\gamma_j)_y$, $(\gamma_j)_{\max}$ and $(\gamma_j)_{res}$ fall within the range of 0.00001-0.0008, 0.0005-0.0017, 0.0012-0.003, 0.0015-0.0037. It was observed that the axial compression load reduced the shear strain values while it increased the joint shear stress. The suggested values for backbone parameters are valid for joints with concrete strength of greater than 60 MPa and steel reinforcements of yield strength of higher than 500 MPa.

Figure 6.9 Backbone curve model for the shear behavior of joints

Table 6.2 Backbone parameters of specimens

Specimen	$(\tau_{jh})_{cr}$	$(\tau_{jh})_y$	$(\tau_{jh})_{max}$	$(\tau_{jh})_{res}$	$(\gamma_j)_{cr}$	$(\gamma_j)_y$	$(\gamma_j)_{max}$	$(\gamma_j)_{res}$	$(\tau_{jh})_{max}$ ASCE41	$(\tau_{jh})_{res}$ ASCE41	$(\gamma_j)_{max}$ ASCE41	$(\gamma_j)_{res}$ ASCE41
IN80	1.5	3.2	5.3	3.3	0.02	0.5	3	3.7	1.245	0.3	15	15
IH80	2.6	3.9	4.5	0.06	0.06	0.5	1.2	1.5	1.245	0.5	15	15
IH80A	2.4	5.1	6.4	2.9	0.04	0.1	0.1	0.3	1.245	0.5	15	15
IN100	2.4	3.1	4.1	1.1	0.8	1.4	2.4	3.6	1.245	0.5	15	15
IH100	1.2	2.5	4.4	1.7	0.2	0.9	1.7	2.8	1.245	0.2	15	15
IH60	0.7	3.8	5.7	0.7	0.2	1.8	1.9	2.1	1.245	0.1	15	15
IH60A	1.4	1.4	5.8	2.9	0.02	0.1	0.02	0.1	1.245	0.3	15	15
EN80	1.7	2.6	3.4	3.3	0.1	0.9	1.2	1.6	0.996	0.3	12	12
EH80	1.2	2.8	4.2	2.3	0.1	0.5	0.9	1.2	0.996	0.2	12	12
EH80A	2.8	3.3	4.2	0.6	0.03	0.03	0.1	0.1	0.996	0.6	12	12
EH60	1.2	1.6	2.1	0.2	0.3	0.6	1.2	1.2	0.996	0.2	12	12

Note: Stress values are in (MPa) and strain values are in (10^{-3} rad)

6.9 Bond Strength of Beam Bars in the Joint Region

The bond condition of specimens is usually related to the ratio of beam and column bar diameters to column and beam depths. **Figure 6.10** shows the local bond stresses along the longitudinal beam bars in the joint region of test specimens. It is observed that IH60 and IH60A with T19 beam bars show higher bond stresses. The bond stress in the specimens with axial loadings are higher than that in similar specimens tested under zero axial loads, which is compatible with the assumptions and recommendations of NZS 3101-06 [N3]. This observation clarifies the beneficial effect of axial loadings on the bond strength of beam bars. However, deterioration in the bond stress of IH60A with a larger beam bar diameter of 19 mm was observed after a DR of 1.5%. Comparing the bond condition of IN80 and IH80, it is concluded that utilizing higher grade longitudinal beam bars will cause a slight decrease in the bond strength. However, test results of IN100 and IH100 showed a reverse trend. Consequently, it is concluded that using HSC will improve the bond condition when HSS reinforcements are used as longitudinal beam bars.

Figure 6.10 Bond stress along beam bars in joints

6.9.1 Evaluation of the average bond stress of beam bars passing through interior beam-column joints

The main challenge of incorporating HSS reinforcements in beam-column joints of moment frames is regarding the anchorage problem of longitudinal beam bars passing through the joint region. Hence, some limitations on diameter of the bars passing through the joint have been suggested by ACI 318-11 [A1] and NZS 3101-06 [N3] as follows:

$$\frac{d_b}{h_c} \leq \frac{1}{20}; (\text{ACI 318-11}) \quad (6.2)$$

$$\frac{d_b}{h_c} \leq 6 \left(\frac{\alpha_t \alpha_p}{\alpha_s} \right) \alpha_f \frac{\sqrt{f'_c}}{\alpha_0 f_y}; (\text{NZS3101-06}) \quad (6.3)$$

Where the value of f'_c is smaller than 70 MPa and α_f and α_d are equal to 1.0 in the case of beam bars in one way frames and in ductile plastic regions.

These limitations in the bar diameter to column depth ratio, $\frac{d_b}{h_c}$, is to assure the acceptable bond performance of joints and its reliability is investigated in this study.

The average bond stress in longitudinal beam bars in a typical interior beam-column joint is shown in **Figure 6.11**. The measured bond stress u_a in the top and bottom beam bars of the specimens in this study and previous research conducted by Li and Leong is presented in **Figure 6.12**. It is observed that the bond performance of the specimens with Grade 700 longitudinal beam reinforcements is satisfactory if the value of u_a is smaller than $0.13\sqrt{f'_c}$ and $0.38\sqrt{f'_c}$ for the top and bottom bars, respectively. It can be suggested that if interior beam-column joints with HSS reinforcements are designed with u_a not larger than the values mentioned above, bond failure of beam bars may not occur and a ductile behavior can be ensured. In addition, test results showed that the average bond stress to preclude bond failure in joints with lower grades of longitudinal beam reinforcement is smaller.

Figure 6.11 Evaluation of average bond stress of top and bottom beam bars in the joint region

Figure 6.12 Bond stress of beam bars

The bond stress of top beam bars can be derived based on the equilibrium of forces shown in **Figure 6.11** as follows:

$$A_s \alpha_0 f_y + C_{st} = n_t \pi d_{b,t} h_c u_{a,t} \quad (6.4)$$

Where n_t is the number of top beam bars and α_0 is the over-strength factor. The ratio of top beam bar diameter to the column depth can be obtained as follows:

$$\frac{d_{b,t}}{h_c} = \frac{4}{\alpha_0} \frac{u_{a,t}}{f_y} \frac{1}{1 + 0.56\beta} \quad (6.5)$$

Where β is the ratio of bottom to top beam bars. A similar procedure can be done for the bottom beam bars:

$$\frac{d_{b,b}}{h_c} = \frac{4}{\alpha_0} \frac{u_{a,b}}{f_y} \frac{1}{1 + 0.56/\beta} \quad (6.6)$$

Based on these equations and the values of u_a obtained in **Figure 6.12**, the suggested $\frac{d_b}{h_c}$ ratios of test specimens and its comparison with ACI 318-11 [A1] and NZS 3101-06 [N3] recommendations is presented in **Table 6.3**.

6.10 The Effect of Varying Column Axial Loading on the Failure Mechanism of Beam-Column Joints

As stated by Paulay and Priestley [P4], column axial loading will improve the performance of beam-column joints by increasing the inclination of the main compression strut. In addition, increasing the axial load level will enhance the bond condition in joints. It is known that the imposed axial loading on columns of moment resisting frames is not constant due to the effects of lateral loadings. However, most of the experimental studies on joints have been conducted without implementing the varying axial loading, which is mainly due to its complexity. It has been observed that the varying axial loading will result in variation in the joint shear strength. When the axial load level is decreased, diagonal tension cracking in the joint region may be observed. However, increasing the axial compression loading will increase the shear force demand in the joint region.

Table 6.3 Bond stress of interior specimens

Specimen	Bar location	Bar size	Concrete grade	Steel grade	Bottom to top bar ratio(β)	$\frac{d_b}{h_c}$ (ACI)	$\frac{d_b}{h_c}$ (NZS)	$\frac{d_b}{h_c}$ (Test)	$\frac{u_a}{\sqrt{f'_c}}$ (ACI)	$\frac{u_a}{\sqrt{f'_c}}$ (NZS)	$\frac{u_a}{\sqrt{f'_c}}$ (Test)
IN80	Top	16	80	564	0.5	1/20.0	1/33.4	1/28.1	1.0	0.6	0.2
	Bottom					1/20.0	1/28.4	1/28.1	1.0	0.7	0.5
IH80	Top	16	80	712	0.5	1/20.0	1/42.1	1/28.1	1.3	0.6	0.1
	Bottom					1/20.0	1/35.8	1/28.1	1.3	0.7	0.4
IH80A	Top	16	80	712	0.5	1/20.0	1/27.5	1/28.1	1.3	0.9	0.3
	Bottom					1/20.0	1/23.4	1/28.1	1.3	1.1	0.6
IN100	Top	16	100	564	0.5	1/20.0	1/29.8	1/28.1	0.9	0.6	0.1
	Bottom					1/20.0	1/25.4	1/28.1	0.9	0.7	0.6
IH100	Top	16	100	712	0.5	1/20.0	1/37.7	1/28.1	1.1	0.6	0.2
	Bottom					1/20.0	1/32.1	1/28.1	1.1	0.7	0.3
IH60	Top	19	60	707	1	1/20.0	1/45.4	1/23.7	1.8	0.8	1.4
	Bottom					1/20.0	1/38.7	1/23.7	1.8	0.9	0.7
IH60A	Top	19	60	707	1	1/20.0	1/39.2	1/23.7	1.8	0.9	0.8
	Bottom					1/20.0	1/33.3	1/23.7	1.8	1.1	0.6

In order to have a better understanding of the influence of varying axial loading on the behavior of beam-column joints and its effect on altering the failure mode, the hierarchy of strength assessment was performed on joints. The procedure was proposed based on axial load-moment (P-M) performance domain. The capacity curves of each element within the beam-column joint are represented based on the conducted section analysis. The evaluation of failure mechanism progression is performed according to demand curves for constant axial load cases as well as the varying axial load scenario. As shown in **Figure 6.13** for the test specimens, the capacities of the beam, column, and joints are evaluated based on the axial load level and the moment value. It is clearly observed that if the demand on specimens is defined based on constant axial loading, the expected failure mechanism will be the beam flexure, which is compatible with test results in the current study. In the case of beam-column joints in lower floors of a multi-span frame, the variation of the axial load level is significantly high. The demand values in this case are shown in **Figure 6.13** as two lines in tensile and compressive loading directions, which are determined based on the varying axial loading pattern. In this case, it is noted that the joint shear failure may occur as the primary failure mechanism, which is not a ductile failure mode. Two different loading patterns of high varying axial loading and low varying axial loading has been illustrated in **Figure 6.13** for Specimens IH60A and IH80A. These loading patterns represent the cases of lower floor joints and upper floor joints in moment frames, respectively. The sequence of the damage pattern in the case of high varying axial loading is observed to be the joint damage, followed by the damage in beams and finally in the column. However, in the case of lower varying axial loading, the damage pattern is beam hinging followed by joint damage and finally the column damage. Under the assumption of constant axial loading, damage in beams will be observed at first followed by the column damage and finally the damage in the joint panel.

Figure 6.13 Hierarchy of strength for the test specimens

6.11 Conclusions

In this chapter, the seismic performance of HSC beam-column joints with HSS reinforcement was evaluated through analytical approaches. Based on this study, following conclusions can be summarized:

1. The stiffness degradation was observed to be less in specimens with HSS reinforcement. The axial compression loading was crucial to increasing the stiffness degradation of specimens and the role of concrete grade observed to be negligible.
2. The γ values of joint shear strength of the specimens were considerably less than the value of $1.245\sqrt{f'_c}$ and $0.996\sqrt{f'_c}$ suggested by ACI-ASCE 352-02 for interior and exterior joints. The joint shear strength value of $\gamma = 7.2\sqrt{f'_c}$ and $\gamma = 5.4\sqrt{f'_c}$ is suggested based on the test results in this study.
3. A constitutive stress-strain relationship with a multi-linear envelope is developed in this study for the joint shear modeling purpose. The model was developed based on test results. The four key points of the backbone curve for the shear response of joints were also defined.
4. The $\frac{d_b}{h_c}$ ratio of test specimens and its comparison with ACI318-11 and NZS3101-06 [N3] recommendations was conducted. In addition, it was concluded that using HSC will improve the bond condition when HSS reinforcements are used as longitudinal beam bars.
5. The hierarchy of strength assessment has been performed on joints. The procedure was proposed based on moment-axial load (M-P) performance domain. The effect of varying axial loading was compared to that of constant compressive loading which was adopted in the experimental program.

CHAPTER 7

FINITE-ELEMENT MODELING OF BEAM-COLUMN JOINTS CONSTRUCTED USING HIGH STRENGTH MATERIALS

7.1 Introduction

The objective of the finite element analysis in the current study is to gain insight on the seismic behavior and nonlinear response of RC beam-column joints. It is more feasible to measure the stress distribution and deformation values of the structural members using FE analysis rather than carrying out an experimental program. Furthermore, FE modeling could be used to study the effect of different design parameters such as the axial loading level, the concrete compressive strength, and the strength of the longitudinal and transverse reinforcement on the behavior of test specimens. The following sections explain the finite-element study as well as the procedures carried out to validate the models using the experimental results. In addition, the effect of different parameters on the seismic behavior of the test specimens is investigated.

A numerical study on RC beam-column joints has been done using the finite element method (FEM). The finite-element computer program DIANA 9.4 [D1] was used in this study. A three-dimensional (3D) model was developed to overcome the shortcomings and simplifying assumptions of 2D models. A 3D simulation will improve the out of plane behavior of the models, which will affect the concrete strength in the primary plane of loading, the rigidity of columns and joints, and so forth. Moreover, 3D simulation provides the opportunity to study different state of stresses in various sections and key points such as inside the joint region. Modeling assumptions and procedure has been presented in the previous research [A4, L4]. However, further explanation on FE modeling is presented in the sections that follow.

7.2 Finite-Element Modeling

A commercial nonlinear finite-element program called DIANA (**Displacement Analyzer**) is used to conduct a 3D nonlinear analysis of the specimens. There is a

huge number of elements and nonlinear constitutive models available in the element and material library of DIANA 9.4 [D1]. A twenty-node isoperimetric solid brick element was used to represent the concrete material. This solid element is based on the quadratic interpolation and Gauss integration. The translation u_i is formulated as the following polynomial function:

$$\begin{aligned}
 u_i(\xi, \eta, \zeta) = & a_0 + a_1\xi + a_2\eta + a_3\zeta + a_4\xi\eta + a_5\eta\zeta + a_6\xi\zeta + a_7\xi^2 + \\
 & a_8\eta^2 + a_9\zeta^2 + a_{10}\xi\eta\zeta + a_{11}\xi^2\eta + a_{12}\xi^2\zeta + a_{13}\xi\eta^2 + \\
 & a_{14}\xi\zeta^2 + a_{15}\eta^2\zeta + a_{16}\eta\zeta^2 + a_{17}\xi^2\eta\zeta + a_{18}\xi\eta^2\zeta + \\
 & a_{19}\xi\eta\zeta^2
 \end{aligned} \tag{7.1}$$

The longitudinal reinforcing bars were modeled by separate truss elements which are connected to concrete elements using interface elements. Transverse steel bars were modeled using the DIANA option of embedded reinforcements.

7.2.1 Constitutive models for the simulation of reinforced-concrete structures

The global and local response of RC structural members depends on the behavior of the composing material. In RC beam-column joints, the applied loading is primarily resisted by the flexural behavior of beams and columns. Hence, the concrete constitutive model should be able to predict the behavior of concrete under the uniaxial and biaxial compression and tension. On the other hand, the reinforcements are subjected to uniaxial stresses as the dowel action of steel bars in beam-column joints are not accounted for. Consequently, the reinforcement constitutive model should be able to capture the uniaxial response in tension and compression. For the interface elements which are connecting the reinforcement to the concrete element, the tangential response should be modeled when choosing the related constitutive relationship.

7.2.1.1 Modeling of concrete

Although concrete is a composite, it is usually modeled as a homogenous material in order to reduce the simulation and computational efforts. In the uniaxial

compression stress level up to the 30% of the concrete compressive strength, concrete will remain in the linear-elastic phase. The reason could be explained as the pre-existing micro cracks are still stable and not propagating. As the uniaxial compression stress is increasing, the nonlinearity could be observed in the load-deflection response of the concrete cylinders under laboratory compression.

A constitutive model for concrete includes the cracking behavior, the behavior in tension, shear behavior, compression hardening and softening, and the lateral confinement influence. The modeling assumption and procedure for each of these factors is described in the sections that follow.

7.2.1.1.1 Concrete cracking behavior

Tensile loading will cause some discrete cracks in the concrete material. When the concrete is subjected to tensile loading of less than 90% of the maximum tensile strength, it will behave similar to a linear-elastic material. When the loading goes beyond 90% of the concrete tensile strength, micro cracks will appear and they will propagate rapidly in a perpendicular direction to the principal stress direction.

The fracture energy, which is a material parameter, is defined as the energy released per unit area of the crack surface. By definition, a crack represents a displacement discontinuity. Cracks can be modeled discretely as done by Ngo and Scordelis [N1]. This method relies on the knowledge about the location and orientation of the cracks, which makes it not feasible to be utilized all the time. As a standard practice, the displacement discontinuities (cracks) in the finite-element approach are modeled in a smeared manner in a specific region with reduced strength and stiffness as introduced first by Rashid [R1]. Using the smeared crack model, a fictitious constitutive model will be adopted to simulate cracking which eliminates the necessity of modifying the geometry and meshing.

In smeared cracking modeling, the available options are multi-directional fixed crack model and total strain crack models. In the multi-directional fixed crack model, cracking cannot be combined with orthotropic material and cracking is specified as a combination of tension cut-off, tension softening and shear retention. In the current research, the total strain rotating crack model will be used. This model is very well suited for Serviceability Limit State (SLS) and Ultimate Limit

State (ULS) analyses which are predominantly governed by cracking or crushing of the material. The constitutive model based on the total strain crack is developed, consistent with the modified compression field theory proposed by Vecchio & Collins [V1]. The three dimensional extension to the theory has been done by Selby and Vecchio [S4]. In total strain crack models, the behavior of the material is modeled as a single stress-strain relationship and other constitutive models cannot be combined with the total strain crack model, consequently. When using the total strain model, two different approaches of “Rotating Crack Model” and “Fixed Crack Model” can be assigned. In the rotating crack model, the stress-strain relationships are evaluated in the principal directions of the strain vector and this is a well suited approach for constitutive modeling of RC structures. In the fixed stress-strain concept, the stress-strain relationships are evaluated in a fixed coordinate system which is fixed upon cracking.

In the total strain cracking model, the properties of the material should be defined in order to simulate the material constitutive relationship accurately. In the following sections, the tensile behavior, shear behavior, compression behavior, and lateral influence of the total strain crack model will be discussed.

7.2.1.1.2 Tensile behavior of concrete

Within the concept of the smeared cracking, the concrete does not behave in a brittle manner when the maximum tensile strength is reached, but exhibits a tensile softening response. Several pre-defined tension softening functions are available in the library of Diana 9.4. **Figure 7.1** shows all the predefined tension softening curves for total strain crack model. The softening function is based on the concrete tensile fracture energy (G_f) and the crack bandwidth (h). The fracture energy is defined as the absorbed energy in creating a fracture surface of unit area, which represents the toughness of the material. Generally, there are several solutions to determine the fracture energy but the most common one is the experimental three-point test on a notched beam conducted by Hillerborg *et al.* [H3], Hillerborg [H2], Malvar and Warren [M2], Malvar and Fourney [M1], and Kozul and Darwin [K2]. The three-point bending test can be performed using the recommendations of RILEM 50-FMC [R6].

Figure 7.1 Predefined tension softening for total strain crack model in DIANA [D1]

As shown in **Figure 7.2**, L is the length between the two supports, h is the height of the specimen, b is the out of plane width of the specimen, t is the notch width, and h_n is the notch height. According to RILEM, a load is applied to the top surface of the specimen to produce a crack with a constant opening rate. The fracture energy (G_f) and the concrete tensile strength (f_t) are determined in order to find the value of the ultimate crack opening (w_u). The tensile strength of concrete (f_t) is calculated according to CEB-FIP model code [C3] as follows:

$$f_t = 0.3 f_c^{2/3} (\text{MPa}) \quad (7.2)$$

Figure 7.2 Three point test schematic [H1]

In the current research, a linear tension softening curve is chosen to represent the post-peak tension behavior of concrete. The values of the parameters such as (G_f) and (f_t) for the finite-element modeling are summarized in **Table 7.1**.

Table 7.1 Material properties of finite-element models

Material Parameters	Values
Concrete tensile strength, f_t	4.6 MPa
Concrete compression strength, f_c'	60 MPa
Modulus of elasticity of concrete, E_c	2×10^4 MPa
Poisson ratio of concrete, ν	0.16
Yield strength of steel in tension, f_y	460 MPa
Ultimate strain of steel, ε_u	0.16
Modulus of elasticity of steel, E_s	2.1×10^5 MPa
Fracture energy, G_t	0.2

7.2.1.1.3 Shear behavior of concrete

The modeling of the shear behavior is required for the total strain fixed crack models in which the shear stiffness of the material is reduced after cracking. In DIANA 9.4 [D1], models are available for constant shear retention, shear retention varying with shear strain, and shear retention based on the damage due to cracking.

The constant shear retention model can be utilized with the total strain fixed crack models. The shear retention factor (β) relates the post-cracking shear stiffness to the pre-cracking initial shear stiffness as follows:

$$G^{cr} = \beta.G \quad (7.3)$$

Figure 7.3 Constant shear retention for total strain crack models [D1]

The shear retention factor (β) is ranging from 0 to 1 ($0 \leq \beta \leq 1$). In variable shear retention models, the points of the shear stress-shear strain diagram should be defined. Since the total strain rotating crack has been adopted in this research, the shear retention factor is assumed to be equal to 1.

7.2.1.1.4 Compression behavior of concrete

The behavior of concrete in compression is a pressure-dependant relationship in which the higher isotropic stress will result in higher strength and ductility of the concrete. In the presence of the lateral confinement, the compressive stress-strain relationship should be modified in order to take into account the effect of the increased isotropic stress. The compressive stress-strain relationship in DIANA is available as several predefined functions as shown in **Figure 7.4**. The predefined compression functions include elastic, ideal, thorenfeldt, linear, multi-linear, saturation type, and parabolic functions.

Figure 7.4 Predefined compression behavior for total strain models in DIANA [D1]

In the current research, a parabolic curve is selected to represent the compressive behavior of concrete. In DIANA, the parabolic curve is formulated based on the fracture energy. According to Feenstra [F1], a compression curve is adopted as shown in **Figure 7.5**.

Figure 7.5 Parabolic compression curve in DIANA [D1]

7.2.1.2 Modeling of reinforcement

In this study, the longitudinal reinforcing bars are assumed to act as separate truss elements. The forces are assumed to be developed along the longitudinal axis of the truss members. In addition, transverse reinforcements are assumed to have a perfect bond with the surrounding concrete. Hence, the embedded reinforcement option in DIANA 9.4 is used for modeling the stirrups and hoops in beams and columns. The reinforcement modeling in DIANA is illustrated in **Figure 7.6**.

The Von Mises yield criterion with isotropic strain hardening was used to characterize the constitutive behavior of the reinforcement. The constitutive behavior of the reinforcement is shown in **Figure 7.7**.

According to the available test data in the literature, the bond deterioration between the longitudinal reinforcing bars and concrete happened during the tests and the bond-slip was observed in longitudinal beam bars. Consequently, the bond-slip behavior of the reinforcing bars is considered in the current FE study and the bond-slip modeling procedure in DIANA is described in the sections that follow.

Figure 7.6 Reinforcement modeling in the finite-element approach

Figure 7.7 Stress-strain relationship for the steel reinforcement

7.2.1.2.1 Bond-slip modeling

The bond-slip characteristic of the reinforcing bars is a crucial part of the modeling of RC elements as the pinching behavior of hysteresis loops is mainly caused by opening and closing of cracks or debonding of the reinforcing bars. **Figure 7.8** shows the bond elements which are incorporated to connect the reinforcement truss elements to the mother concrete elements. Concrete elements are treated as continuum 3D elements in the formulation, while the truss and bond elements are supposed to have constant strain and constant slip.

As discussed, concrete and reinforcing steel materials have been modeled in DIANA using twenty-node isoparametric solid brick elements and three-node numerically integrated truss elements, respectively. Consequently, the bond elements should be interface elements between a quadratic line and a quadratic brick solid element in a three-dimensional configuration. A 3+20 node, line-solid connection element called CHX69I is adopted in this study as shown in **Figure 7.8**. The element is based on quadratic interpolation.

Figure 7.8 Material elements in the finite-element approach [D1]

For the modeling of bond behavior between the concrete and reinforcing steel, a bond response should be defined to correlate the bond stress to the slip values. The bond-slip law presented by CEB-FIP Code 1990 [C1] is calibrated based on an empirical model by Eligehausen as shown in **Figure 7.9**.

Figure 7.9 Bond-slip law by CEB-FIP Code 1990 [C1]

Based on Eligehausen *et al.* [E3], the maximum bond stress is taken as 2.64 times the square root of the concrete compressive strength (MPa), while the residual stress is defined as 0.9 times the square root of the concrete compressive strength (MPa). The slip levels are also determined based on the tests conducted by Eligehausen. The slip values at the maximum stress level are nominated as s_1 and s_2 and they are defined as $s_1 = 1$ mm and $s_2 = 3$ mm in this study. The residual stress is assumed to develop at a slip value of 10.5 mm.

The ascending and descending parts of the bond-slip curve are formulated as below:

$$\tau = \tau_{\max} \left(\frac{s}{s_1} \right)^\alpha ; 0 \leq s \leq s_1 \quad (7.4)$$

$$\tau = \tau_{\max} ; s_1 \leq s \leq s_2 \quad (7.4b)$$

$$\tau = \tau_{\max} - (\tau_{\max} - \tau_f) \frac{s - s_1}{s_3 - s_2} ; s_2 \leq s \leq s_3 \quad (7.4c)$$

$$\tau = \tau_f ; s_3 \leq s \quad (7.4d)$$

The bond response for various levels of the concrete compressive strength which were used in the current FE study is shown in **Figure 7.10**.

Figure 7.10 Bond-slip response for various concrete grades

7.2.2 Type of elements

A twenty-node isoperimetric solid brick element as shown in **Figure 7.8(a)** was used for the modeling of concrete. This solid element is based on the quadratic interpolation and Gauss integration. General meshing of numerical models in DIANA is shown in **Figure 7.11** and a typical principal compressive stress distribution in specimens is illustrated in **Figure 7.12**.

Figure 7.11 Meshing of the finite-element models

7.2.3 Solution algorithm

The Newton-Raphson method was applied to solve the nonlinear equations. In the regular Newton-Raphson method, the stiffness relation is evaluated in each iteration step. This method converges to the final answer within a few numbers of iterations as a quadratic convergence characteristic is shown by this method. Generally, the Newton-Raphson method needs a few iterations to converge, but each iteration step needs quite a long time to be done. Regular Newton-Raphson iteration is shown in **Figure 7.13**.

Figure 7.12 Principal compressive stress distribution in Specimen IH100

Figure 7.13 Regular Newton-Raphson iteration [D1]

On the other hand, the Quasi-Newton method (Secant method) uses the information of previous solution vectors and out-of-balance force vectors in order to come up with a better approximation of the solution. Iteration steps in this method are performed much faster compared to the regular Newton-Raphson method, which is desirable for the parametric investigation in this study. Hence, the Quasi-Newton iteration method is used in the current study to make a balance between the solution accuracy and the required computational effort. The Secant iteration method is shown in **Figure 7.14**.

Figure 7.14 Quasi-Newton iteration [D1]

7.3 Validation of Finite-Element Models

Upon the completion of finite-element modeling, the FE result and predictions are to be verified based on the available test results. An extensive parametric study will be conducted after the validation of FE models. The verification process is performed based on the comparison between the hysteresis loops obtained from FE simulations and the experimental test results. In addition, the compressive stress distribution in different parts of the specimens as well as the stress values along steel reinforcement is also verified.

The following sections present the comparison between the FE results and test observations of the current study and the tests conducted by Li and Leong [L2].

7.3.1 Validation of FE models using the current experimental results

7.3.1.1 Hysteretic behavior

Figure 7.15 shows the hysteresis loop of the story shear (kN) versus the lateral displacement (mm) where FEM result is shown as dotted line. Numerical predictions show very satisfying accuracy almost in all cycles regarding the maximum strength value and the predicted pinching behavior. The general trend of the hysteresis loops predicted by FE modeling is similar to that of test results. As observed, the FE simulation successfully predicts the energy dissipation capacity of the specimens where the values obtained from the FE analyses is close to the test results.

7.3.1.2 Distribution of concrete compressive stress

The stress contour during the peak of DR= 4.0% (the last cycle of lateral loading) is depicted in **Figures 7.16** through **7.27** for the interior and exterior joints. In general, the joint core was highly stressed at the end of the simulation after the peak lateral load. As illustrated, the compression strut has been formed at adjacent corners of the joint core, which is in agreement with the observed test result. The formation of plastic hinges is clearly predicted in FE simulations which is consistent with experimental observations.

Figure 7.15 FE verification of global behavior of test specimens

Figure 7.16 Evolution of the compressive stress in Specimen IN80

Figure 7.17 Evolution of the compressive stress in Specimen IH80

Figure 7.18 Evolution of the compressive stress in Specimen IN100

Figure 7.19 Evolution of the compressive stress in Specimen IH100

Figure 7.20 Evolution of the compressive stress in Specimen IH60

Figure 7.22 Evolution of the compressive stress in Specimen IH80A

Figure 7.23 Evolution of the compressive stress in Specimen EN80

Figure 7.24 Evolution of the compressive stress in Specimen EH80

Figure 7.25 Evolution of the compressive stress in Specimen EH60

Figure 7.26 Evolution of the compressive stress in Specimen EH60A

Figure 7.27 Evolution of the compressive stress in Specimen EH80A

7.3.1.3 Distribution of tensile stress in steel reinforcement

Figures 7.28 through **7.39** show the stress values in steel reinforcement of the models in a DR of 1.0% and 2.0%. As observed, longitudinal beam bars have reached their yield strength and the stress values are critical in a beam region near the column face. On the other hand, the transverse reinforcement stress values are lower than the yielding value, especially in the joint region. The observed stress values along the steel reinforcement are in strong agreement with those attained in the experimental program.

Both the experimental results and FE simulations show that the column section has remained elastic. This is expected as the specimens have been designed according to the strong-column and weak-beam philosophy. In general, stress values along the longitudinal and transverse steel reinforcement and also the failure mechanism of the specimens are similar to experimental findings.

7.3.1.4 Energy dissipation capacity

The comparison between the experimental and FE results for the strain energy of the specimens is shown in **Figure 7.40**. As a general trend, the FE predictions underestimate the energy dissipation capacity at the initial stages of the test. However, the difference between the experimental results and FE predictions becomes less significant in higher drift ratios and the FE results overestimate the energy dissipation of several specimens at the final loading cycles. The overall FE predictions show acceptable correlation to the experimental observations.

Figure 7.28 Evolution of the stress along reinforcement in Specimen IN80

Figure 7.29 Evolution of the stress along reinforcement in Specimen IH80

Figure 7.30 Evolution of the stress along reinforcement in Specimen IN100

Figure 7.31 Evolution of the stress along reinforcement in Specimen IH100

Figure 7.32 Evolution of the stress along reinforcement in Specimen IH60

Figure 7.33 Evolution of the stress along reinforcement in Specimen IH60A

Figure 7.34 Evolution of the stress along reinforcement in Specimen IH80A

Figure 7.35 Evolution of the stress along reinforcement in Specimen EN80

Figure 7.36 Evolution of the stress along reinforcement in Specimen EH80

Figure 7.37 Evolution of the stress along reinforcement in Specimen EH60

Figure 7.38 Evolution of the stress along reinforcement in Specimen EH60A

Figure 7.39 Evolution of the stress along reinforcement in Specimen EH80A

Figure 7.40 Comparison of strain energy between FE and experimental results

7.3.2 Validation of FE models using the experimental results of the specimens tested by Li and Leong (2014)

Li and Leong [L2] performed a series of tests on eight interior beam-column joints. The concrete compressive strength in all of the specimens is 60 MPa, while the yield strength of longitudinal bars is 500 MPa. The yield strength of transverse reinforcement in beams and columns is 360 MPa. The diameter of reinforcing bars is ranging from 12 mm to 28 mm in different specimens. Specimens of NS-series were tested with zero axial loading, while the column axial compression loading of $0.3 f'_c A_g$ was applied to the specimens of AS-Series. Description of the design and detailing of the test specimens is shown in **Table 7.2** and the properties of the steel reinforcement are mentioned in **Table 7.3**.

Table 7.2 Summary of the specimens tested by Li and Leong [L2]

Specimen		NS1 / AS1	NS2 / AS2	NS3 / AS3	NS4 / AS4
Beam	Size	250 x 500 mm			
	Top bar	7T12 ($\rho = 0.73\%$)	4T16 ($\rho = 0.71\%$)	4T16 ($\rho = 0.71\%$)	2T20 ($\rho = 0.56\%$)
	Bottom bar	7T12 ($\rho' = 0.73\%$)	2T16 ($\rho' = 0.36\%$)	4T16 ($\rho' = 0.71\%$)	2T16 ($\rho' = 0.36\%$)
	Stirrups	R6@75 & R6@150	R10@90 & R6@150	R10@90 & R6@150	R10@90 & R6@150
Column	Size	450 x 300 mm			
	Main bar	4T28+2T24+2T20 ($\rho_t = 1.40\%$)	4T24+4T16 ($\rho_t = 0.92\%$)	T32+4T20 ($\rho_t = 1.60\%$)	4T28+4T16 ($\rho_t = 1.29\%$)
	Hoops	2R10@60 & 2R10@120	(R10+R6)@60 & (R10+R6)@120	2R10@90 & 2R10@120	2R10@90 & 2R10@120
Joint	Hoops	5x2R10@65	(5R10+5R6)@65	5x2R10@65	R10@65

Table 7.3 Properties of the reinforcement in specimens tested by Li and Leong [L2]

Bar Size	Yield Strain, $\varepsilon_y (10^{-6})$	Yield Strength, $f_y (MPa)$
6	1766	362
10	1727	354
12	2488	510
16	2478	508
20	2502	513
24	2522	517
28	2468	506

Figure 7.41 shows the final cracking pattern of the experimental specimens tested by Li and Leong. The comparison of the FE simulation results and the experimental observations of the specimens is presented in the following sections to validate the FE approach in the current study.

7.3.2.1 Hysteretic behavior

Figure 7.42 shows the hysteresis loop of the story shear (kN) versus the lateral displacement (mm), where the FE results are shown in dotted and experimental results are shown in solid lines. The FE analysis results show satisfying accuracy almost in all loading cycles regarding the maximum strength and the pinching behavior. The general trend of the hysteresis loops from the FE analysis is similar to the test result.

The FE simulation successfully predicts the energy dissipation capacity of the specimens. However, the FE results are conservative in the last cycle of the lateral loading where FE simulation tends to estimate a lower energy dissipation capacity. As the specimens were designed based on the strong-column and weak-beam philosophy, the column reinforcing bars should remain elastic. The FE results verify this assumption as shown in **Figure 7.44**, where the maximum stress in column longitudinal bars is smaller than the yielding strength. In addition, it is observed that the central column bars experienced lower stress values compared to the side bars. The respective strain values measured at central bars were much smaller than those of the side bars.

Figure 7.41 Crack pattern at the final stage of loading for the specimens tested by Li and Leong [L2]

Figure 7.42 Global verification of FE models using the specimens tested by Li and Leong

7.3.2.2 Distribution of concrete compressive stress

The concrete compressive stress distribution in specimens at a DR of 4.0% is presented in **Figure 7.43**. In general, the joint core was highly stressed at the final loading stage. As illustrated, the compression strut has been formed at adjacent corners of the joint core, which is in agreement with the observed test result.

7.3.2.3 Distribution of tensile stress in steel reinforcement

Figure 7.44 shows the stress distribution along the reinforcing bars in the beams, columns, and joints at a DR of 4.0%. As observed, longitudinal beam bars have reached their yield strength and the stress values are more critical in a beam region near the column face. It can be concluded that increasing the yield strength of longitudinal beam bars can directly affect the behavior of the beam-column joints. On the other hand, the stress values of transverse reinforcements are lower than the yielding values. Hence, the influence of the spacing of shear reinforcements as well as the effect of the yield strength of the reinforcing bars is required to be further investigated.

Figure 7.43 Compressive stress distribution in specimens tested by Li and Leong

Figure 7.44 Stress distribution of the reinforcements in specimens tested by Li and Leong

7.4 Parametric Studies on Seismic Behavior of Beam-Column Joints

In this section, a parametric study is carried out in order to investigate the effect of various design parameters on the seismic behavior of beam column joints constructed using high-strength material. Generally, design equations in ACI 318-11 [A1] are derived based on recent experiments done mostly on the specimens constructed using normal-strength materials. As the popularity of using HSC with HSS reinforcements in the construction industry is increasing, it is vital to study the effect of utilizing such material on the seismic behavior of frames and members.

The FE models in this parametric study are three-dimensional beam-column joints with the geometry and reinforcement details as mentioned earlier in this chapter. Hence, the effect of design parameters such as the axial loading ratio as well as the material properties, including the concrete compressive strength and the yield strength of longitudinal and transverse reinforcing bars in beams and columns, are considered. In addition, the influence of using various beam-bar diameter to column-depth ratios in interior joints as well as the effect of development length of beam bars in exterior joints are investigated.

Table 7.4 summarizes all the parameters investigated in the numerical parametric study.

Table 7.4 Investigated parameters in the parametric study

Concrete compressive strength, f'_c (MPa)	50 to 100
Yield strength of steel reinforcement, f_y (MPa)	460 to 930
Yield strength of transverse reinforcement, f_{yt} (MPa)	460 to 930
Axial load ratio, $P/f'_c A_g$	0 to 0.6
Beam bar diameter to column depth ratio, d_b/h_c	1/41 to 1/19
Beam bar development length to column depth ratio, l_{dh}/h_c	1/36 to 1/12

7.4.1 Influence of axial load levels on seismic behavior of joints

Figures 7.45 through **7.49** show the influence of different levels of axial loads on the behavior of Specimens IH80, IH100, EH60, EN80 and EH80. The applied column axial load varied from $0.2 f_c' A_g$ to $0.6 f_c' A_g$ as compared with zero axial load cases. As observed, incorporating axial load levels of up to $0.3 f_c' A_g$ on test specimens caused an enhancement in the strength of specimens by around 16% and 9% for interior and exterior joints, respectively. FE parametric studies showed that the energy dissipation capacity of the specimens with concrete grade of 80 MPa reached its maximum in the case of axial load level of $0.4 f_c' A_g$. However, the highest energy dissipation capacity of IH100 occurred in the case of axial load level of $0.6 f_c' A_g$, which shows the crucial confinement effect of axial loading in the case of higher concrete grades.

In terms of the peak-to-peak stiffness of the models, it is observed that the highest stiffness belongs to the models with the highest axial load levels in early loading stages. In contrast, the models with the high axial load level of $0.6 f_c' A_g$ were observed to have the minimum stiffness at the final loading cycles, which indicates the high stiffness degradation of the specimens while subjected to high level of axial loadings. Although the models with zero axial loads had the lowest stiffness in the early loading stage compared with others, they were observed to have the lowest stiffness degradation throughout the loading cycles.

Figure 7.45 The influence of axial load ratio by FEM in Specimen IH80

Figure 7.46 The influence of axial load ratio by FEM in Specimen IH100

Figure 7.47 The influence of axial load ratio by FEM in Specimen EH60

Figure 7.48 The influence of axial load ratio by FEM in Specimen EN80

Figure 7.49 The influence of axial load ratio by FEM in Specimen EH80

7.4.2 Influence of beam flexural reinforcement grade on seismic behavior of joints

A potential benefit of using HSS reinforcements in beam-column joints of the moment frames is to resolve the steel congestion problem. In this section, the objective is to determine the effect of utilizing HSS reinforcements while the beam bar diameters are reduced and the nominal moment capacity of beams is remaining constant. **Figures 7.50** and **7.51** show the analysis results of finite-element models with various longitudinal beam bars yield strength f_y , while the value of $A_s f_y$ was kept constant for all of them. It is observed that although the value of $A_s f_y = 400KN$ is the same in all of the models, story shear force is reduced by around 10% in interior and 14% in exterior joints when longitudinal beam bars with $f_y = 883MPa$ are used compared with that of $f_y = 303MPa$. In terms of the energy dissipation capacity, the performance of the models is quite similar before $DR=1.0\%$. However, there is a continuous loss of the maximum energy dissipation capacity after a $DR=1.0\%$ as the yield strength value f_y is going higher. At a DR of 4.0%, the models with $f_y = 883MPa$ shows a 44% smaller energy dissipation capacity compared to that in the model with $f_y = 303MPa$. Considering the effect of f_y on the secant stiffness of the specimens, **Figures 7.50** and **7.51** show that utilizing higher grade longitudinal beam reinforcements will reduce the stiffness of the specimens in early loading cycles. However, after a DR of 2.0% all the specimens showed close stiffness values.

Figure 7.50 The influence of the yield strength of beam bars in Specimen IH60

Figure 7.51 The influence of the yield strength of beam bars in Specimen EH80

7.4.3 Determination of beam bar diameter to column depth ratio in interior joints

In Chapter 21 of ACI318-11, the requirements for design of RC members in the inelastic range of response without severe deterioration in strength have been presented. However, in Section 21.1.1.8 it is specified that the design of structural systems not satisfying the requirements mentioned in Chapter 21 is permitted only if experimental and analysis evidences show that the proposed system will have adequate strength and toughness. The minimum experimental evidence that shall be provided in order to validate the use of HSS reinforcements in HSC beam-column joints could be obtained from “Acceptance Criteria for Moment Frames Based on Structural Testing (ACI T1.1-01)” [A2] standard. Due to the inadequate number of experimental data, the proposed analytical model in this study was used to evaluate the performance of HSC joints with HSS reinforcements. **Figure 7.52** shows the variation of energy dissipation capacity versus the column depth to beam bar diameter ratio obtained from the numerical analyses. The minimum energy dissipation value suggested by ACI T1.1 [A2] is shown as dashed lines. Zero energy dissipation of the models is related to the bond failure in joints. It was concluded that the models would achieve the minimum energy dissipation capacity suggested by ACI T1.1 [A2] if the following equation is satisfied:

$$h_c/d_b = f_y^{1.3} / (16\alpha_p \sqrt{f'_c}) \text{ (MPa)} \quad (7.5a)$$

$$h_c/d_b = f_y^{1.3} / (842\alpha_p \sqrt{f'_c}) \text{ (Psi)} \quad (7.5b)$$

Where $\alpha_p = 0.95 + \frac{1}{2} \frac{P}{f'_c A_g}$ is the modification factor for the effect of compression

loading. The comparison between the proposed equations for the joint depth to beam bar diameter ratio and the ratios suggested by ACI 352-02 [A3] and NZS 3101-06 [N3] is presented in **Figure 7.53**. The comparisons have been made for the axial load levels of 0 and $0.3 f'_c A_g$ and the steel reinforcements yield strength of 500, 600 and 700 MPa. It is clearly observed that the bond condition is enhanced when using the higher concrete grade and also in the case of applied axial loadings. It was illustrated that the proposed equation and the NZS 3101-06 [N3]

recommendation provide a close value of h_c/d_b ratio when 500 MPa steel bars are used. However, the difference of these ratios increases as the grade of steel is going higher, which indicates that the current code provisions are not able to address higher bond forces in case of using HSS reinforcements.

Figure 7.52 The effect of beam bar diameter to column depth ratio in interior joints

Figure 7.53 Comparison of the proposed development length for interior joints

7.4.4 Determination of development length of beam bars in exterior joints

The required development length of beam bars specified by ACI 318-11 [A1] and ACI 352-02 [A3] are compared to the specimens design development length in **Table 7.5**. The specimens with T19 beam reinforcements satisfied the development length requirements; whereas Specimens EH60 and EH60A with T19 beam reinforcements did not satisfy the compression development length specified by ACI 318-11 [A1]. Nevertheless, the performance of the specimens proved to be satisfactory. In this section, a finite-element study is used to find an appropriate expression to determine the minimum development length required in exterior beam-column joints. **Figure 7.54** shows the variation of energy dissipation capacity versus the development length to beam bar diameter ratio obtained from numerical modeling. The minimum energy dissipation value suggested by ACI T1.1-01 [A2] is shown as dashed lines. Zero energy dissipation of the models is related to the bond failure in joints. It was concluded that the models would achieve the minimum energy dissipation capacity suggested by ACI T1.1-01 [A2] if the following equation is satisfied:

$$\frac{l_{dh}}{d_b} = \frac{f_y}{4.5\sqrt{f'_c} \left(0.95 + 0.35 \frac{P}{f'_c A_g} \right)} \quad (\text{MPa}) \quad (7.6)$$

Where $\alpha_p = 0.95 + 0.35 \frac{P}{f'_c A_g}$ is the modification factor for the effect of axial

compression loading. This novel expression is to be used to determine the development length of the longitudinal beam reinforcements passing through the exterior joints in case of 90 degree hook bars. The equation has been proposed based on FE analysis on models with concrete grade of higher than 60 MPa and reinforcing steel yield strength of greater than 500 MPa.

Figure 7.55 illustrates the comparison between the required development length of longitudinal beam bars in the joint region (l_{dh}/d_b) suggested by ACI318-11 and proposed in the current study. It is observed that the required development length is increased by 40% when the reinforcement yield strength is increased from

$f_y = 500\text{MPa}$ to $f_y = 700\text{MPa}$. In order to overcome this issue, using high-strength concrete (HSC) is a key solution. Utilizing HSC coupled with HSS reinforcement will result in a 23% reduction in the required development length. In addition, the beneficial effect of column axial loading which is due to its confining effect on the joint is obvious. The required development length is reduced between 17% to 20% when the column axial loading is increased from 0 to $0.6 f'_c A_g$. As a general conclusion, the required development length ratio (l_{dh}/d_b) in case of using HSS reinforcement could be controlled if the effect of column axial loading and using HSC is incorporated.

Table 7.5 Development length of specimens

Specimen	EN80	EH80,EH80A	EH60,EH60A
Design development length ratio(l_{dh}/d_b)	23.1	23.0	19.4
Required tension development length ratio by ACI 318-11 [A1]	11.4	14.5	16.7
Required compression development length ratio by ACI 318-11 [A1]	15.4	19.6	22.6
Required development length ratio by ACI 352-02 [A3]	12.4	15.8	18.2

Figure 7.54 The effect of the development length of beam bars in exterior joints

Figure 7.55 Comparison of the proposed development length for exterior joints

7.5 Conclusions

Nonlinear finite-element approach was obtained in this chapter to further investigate the seismic behavior of the tested beam-column joints. According to this FE study, the following conclusions may be drawn:

1. The verification of FE numerical results against the experimental results of all the twelve interior and exterior specimens showed good agreement in the global and local behavior of specimens. The global verification was conducted on the hysteresis behavior and energy dissipation capacities. The local verification included the compressive stress in specimens and the stress distribution along the reinforcing bars.
2. An extensive parametric study was conducted using the verified FE models. The effect of material properties and the effect of column axial loading presence were investigated. In addition, the influence of using various beam bar diameter to column depth ratios in interior joints as well as the effect of development length of beam bars in exterior joints were investigated.
3. It was observed that incorporating axial load levels of up to $0.3 f'_c A_g$ on test specimens caused an enhancement in the strength of specimens by around 16% and 9% for interior and exterior joints, respectively. In addition, the crucial confinement effect of axial loading in the case of higher concrete grades was shown.
4. The lateral force capacity is reduced in the order of 10% when the normal-strength longitudinal steel reinforcements in beams are substituted by the high-strength ones with the constant value of $A_s f_y$.
5. A novel recommendation for the joint depth to beam bar diameter ratio for interior joints was presented based on FE investigations. Comparison was made between the proposed equation and those suggested by ACI 318-11 [A1] and NZS3101-06 [N3].
6. An equation was proposed to determine the development length of the longitudinal beam bars passing through the exterior joints in case of 90 degree hook bars. Comparison was made between the proposed equation and the ACI 318-11 [A1] recommendations for the tension and compression develop length.

CHAPTER 8

DRIFT CAPACITY OF HSC COLUMNS WITH MIXED GRADE LONGITUDINAL REINFORCEMENT

8.1 Introduction

In the contemporary seismic design of reinforced concrete (RC) frame buildings, the beam sidesway mechanism has formed the basis of the commonly-used ductile design approach. In this mechanism, the most crucial design issue is the flexural yielding of the RC columns. To achieve a proper design, all the loading scenarios such as the dynamic effects of higher mode vibrations and bidirectional earthquake loading circumstances should be considered. As a result, columns are required to have uneconomically large cross sections even if high-strength concrete (HSC) is used. This issue is considerably more severe in high-rise buildings or in frames with long span beams. In order to overcome this problem, a designer can come up with two solutions. First, columns can be designed to be as strong as required by using high-strength material such as high-strength steel (HSS) reinforcement in addition to HSC. In this method, other factors such as the anchorage problem of HSS reinforcement and the concern about the adequate drift capacity of RC columns should be considered prudently. The other method is to establish an appropriate mechanism other than beam sidesway. A favorable mechanism could be the one in which column hinges are observed in all stories but no soft story is developed. In this case, columns are required to have adequate flexural stiffness even after yielding of longitudinal reinforcements so that the lateral stiffness of the stories would not degrade excessively during seismic excitations. Conventional design methods like confinement of core concrete will not guarantee such an ideal flexural behavior of RC columns since the ductility demand may be excessive due to plastic rotation of the column.

The objective of this chapter is to investigate the effectiveness of a column design method which uses longitudinal reinforcement with various steel grades [W1]. Through an experimental program and a series of analytical models, new drift-based

design expressions and charts are proposed. These models can be used as a design tool for (HSC) columns reinforced with mixed grade longitudinal steel bars.

8.2 Validation of Numerical Models

The following sections present the 3D Nonlinear Finite-Element Analysis (NLFEA) carried out on RC columns using computer program DIANA 9.4 [D1]. FE analysis results were compared to those obtained from previous experimental programs in the literature for the verification purpose.

The experimental result of six columns tested by Sato *et al.* [S2] is presented in Section 8.2.1. In addition, the experimental hysteresis results of three column specimens tested by Satyarno [S3] are also studied in Section 8.2.2.

8.2.1 Validation of FE models using the experimental results of the specimens tested by Sato *et al.* (1993)

Six half-scale cantilever RC columns were constructed and tested by Sato *et al.*[S2]. Each test unit has a square cross section of 400 mm × 400 mm as shown in **Figure 8.1**. Shear span length for all the specimens is 1000 mm and the ratio of the column height to depth is 2.5. The longitudinal reinforcement was from 20 mm diameter deformed bar with yield strength of 430 MPa shown as HD20 and 22 mm diameter deformed bar with yield strength of 930 MPa shown as UHD22. Two overlapping square spiral hoops with 80 mm spacing were used in column specimens as transverse reinforcement. These square spiral hoops were from 9.2 mm diameter deformed bars with the yield strength of 1275 MPa shown as U9.2. The axial loading level of columns was $0.3 f'_c A_g$ for Units 1 to 3 and $0.6 f'_c A_g$ for Units 4 to 6. The design concrete compressive strength was 60 MPa for all the specimens. The mechanical properties of reinforcements are shown in **Table 8.1**.

Figure 8.2 illustrates the comparison between the hysteresis response of test specimens through the experimental observation and the FE numerical prediction. The theoretical ultimate loads were calculated using stress-strain relationships for the longitudinal reinforcements and the ACI 318-11 [A1] rectangular compressive stress block for concrete. It is observed that the maximum lateral loading capacity obtained from the analytical approach is almost equivalent to the experimental

values of the ultimate loads in all the specimens. There was agreement with regards to the overall global behavior between the experimental and analytical results. In order to verify the local behavior of numerical models, the comparison has been made between the experimentally obtained strain profiles and the predicted values of strains in FE analyses as illustrated in **Figure 8.3**.

Table 8.1 Mechanical properties of steel reinforcement

Type	Nominal Area, (mm^2)	Yield Strength, (MPa)	Ultimate Strength, (MPa)	Young's Modules, (MPa)
HD20	314	442	590	200000
UHD22	387.1	1033	1165	198700
U9.2	64	1368	1462	201200

Figure 8.1 Details of the testing program

Figure 8.2 Comparison of hysteretic behavior between the experimental and FE results

Figure 8.3 Strain profiles of longitudinal bars in Unit 2

8.2.2 Validation of FE models using the experimental results of the specimens tested by Satyarno *et al.* (1993)

Three cantilever columns were constructed and tested by Satyarno *et al.* [S3]. Dimensions and reinforcement detailing of the specimens were similar to the specimens tested by Sato *et al.* [S2]. Concrete compressive strength of all the specimens was 50 MPa. The longitudinal reinforcement was from 20 mm diameter

deformed bar with yield strength of 430 MPa and from 22 mm diameter deformed bar with yield strength of 1050 MPa. Two overlapping square spiral hoops with 80 mm spacing were used in all the column specimens. The square spiral hoops were from 9.2 mm diameter deformed bar with yield strength of 1250 MPa. The axial column compressive loading of $0.6 f'_c A_g$ was applied to all units.

Figure 8.4 illustrates the comparison between the hysteresis response of test specimens through the experimental observation and the FE numerical prediction. It is observed that the maximum lateral loading capacity obtained from the analytical approach is almost equivalent to the experimental values of the ultimate loads in all the specimens. There was agreement with regards to the overall global behavior between the experimental and analytical results.

Figure 8.4 Experimental and analytical horizontal load-displacement hysteresis loop for the specimens tested by Satyarno [S3]

8.3 Parametric Studies on Seismic Behavior of HSC Columns

After verifying the numerical models against the available experimental results, an extensive parametric investigation is performed to further investigate the seismic behavior of HSC columns reinforced with HSS reinforcements. The following sections present the application of FE modeling technique to study the influence of critical design parameters, such as the axial loading level, shear span ratio and various grades of concrete and steel reinforcement.

8.3.1 Influence of column axial loading on seismic behavior of HSC columns

The influence of the axial loading level was investigated using a finite-element approach. The same horizontal loading history as those applied in the experimental program of the specimens was applied to the columns tip of the numerical models. **Figure 8.5** shows the lateral loading capacity versus horizontal displacements for different levels of applied axial loading on RC column models with different detailing and characteristics. The applied column axial loading varied from $0.1 f'_c A_g$ to $0.6 f'_c A_g$ as compared with zero axial loading cases. As observed, when the column axial loading is increased from $0.3 f'_c A_g$ to $0.6 f'_c A_g$ in models with concrete grade of 60 MPa and mixed grade longitudinal reinforcement, lateral loading capacities are increased around 5%. In models with higher concrete grade of 80 MPa, the same increase in the axial loading level will cause the lateral loading capacity to rise around 15%, which shows the remarkable effect of column axial loading on lateral loading capacity in models with higher concrete grades. Columns with axial loading level of $0.6 f'_c A_g$ showed a higher lateral loading capacity of about 40% and 60% for concrete grades of 60 MPa and 80 MPa as compared with the cases of zero axial loading.

Investigating the behavior of RC columns with various ratios of longitudinal HSS reinforcing bars indicated that the effect of altering axial loading level is identical for HSS bar ratios of $\rho'_t = 30, 50$ and 100%.

The preceding discussions clearly show that incorporating the axial loading levels of up to $0.6 f'_c A_g$ on test specimens will cause an enhancement in strength of the specimens irrelevant of other parameters such as reinforcement detailing and material properties.

Figure 8.5 The influence of the axial load level on the behavior of RC columns

8.3.2 Influence of shear span ratio on seismic behavior of HSC columns

The behavior of slender columns, which is the topic of study in this thesis, is predominantly controlled by flexural response and the truss mechanism remains the most portion of the shear capacity.

In this section, the performance of RC columns with various shear span ratios is studied based on FE analysis results. **Figure 8.6** shows the lateral loading capacity versus horizontal displacements for RC columns with different shear span ratios ranging from 2.5 to 5.0. The shear span ratio of 2.5 is near the lower end of L/h value used in the practice. A complete set of analyses was also performed for the shear span ratios of 3.5 and 5.0, which represent the higher end of the range used in practice. It is clearly shown that columns with larger shear span ratios display smaller moment capacities and higher displacement ductility. Parametric study results indicated that decreasing the shear span ratio of a column from 5.0 to 2.5 will increase the maximum lateral loading capacity between 60 to 85%. It is also observed that the mentioned capacity increase is larger in models with higher axial loading levels. The influence of shear span ratio is observed to be more crucial in sections with minimum ratios of HSS reinforcement. In addition, FE analyses show that the influence of shear span ratio becomes less critical as the yield strength of the longitudinal reinforcement is going higher. However, altering the concrete strength did not have any significant influence on the effect of the column shear span ratios.

Figure 8.6 The influence of shear span ratio on the behavior of RC columns

8.3.3 Influence of mixed-grade longitudinal reinforcement ratio on seismic behavior of HSC columns

In the current study, the emphasis was put on improving the flexural behavior of columns by arranging different detailing of longitudinal reinforcement with various steel grades. Hence, the performance of the columns is studied in terms of the moment capacity and drift capacity. The basic concept in this design method is to observe the longitudinal reinforcements yielding initiating from normal-strength steel bars followed by HSS bars, in the case of increasing displacement demand on the structural member. As a result, flexural strength of the column is expected to increase after yielding of the lowest grade longitudinal reinforcement and the trend will continue until the yielding of HSS bars. In order to quantify the influence of various mixed grade reinforcement detailing, FE parametric studies were performed and lateral loading capacity versus horizontal displacements is presented in **Figure 8.7**. It is observed that in HSC columns with HSS reinforcements, in a section with the HSS bar ratio of 30%, the lateral loading capacities are between 12 to 24% higher than that in a section without HSS bars. This value is between 17 to 36% and 43 to 66% for a section with HSS ratio of 50% and 100%, respectively. It is observed that the influence of altering the ratio of HSS bars is reduced in the cases with higher axial loading levels. In addition, the effect of varying longitudinal steel reinforcement is more significant if the grade of steel is higher. Furthermore, the column drift capacity, which is defined as the ultimate displacement capacity divided by the column height, is related to the grade and ratio of HSS reinforcing bars in the section. Further study on drift capacity of RC columns will be presented in this chapter.

Figure 8.7 The influence of mixed grade longitudinal reinforcement ratios on the behavior of RC columns

8.4 The Estimation of Drift Capacity of HSC Columns

The correlation between lateral drift capacity and concrete confinement in normal-strength columns has been studied in previous research [S1, P6]. However, the confinement requirement in ACI 318-11 [A1] is not based on lateral drift demand and the design equations have been developed based on axial deformability. Hence, drift demand has not been introduced as a design parameter in confinement criteria requirement.

Several studies have proposed drift-based design equations for RC columns. However, the effect of utilizing HSS longitudinal reinforcement and the ratio of high-strength to normal-strength reinforcement has not been incorporated into design equations. Proposed equations by Saatcioglu and Razvi [S1] and Brachmann *et al.* [B3] for normal-strength RC columns are presented as follows:

$$D_{ult} = 0.07 \rho_c \frac{f_{yh}}{f'_c \left(\frac{A_g}{A_c} - 1 \right)} \left(\sqrt{k_2} \frac{P_0}{P} \right) \quad (8.1a)$$

$$D_{ult} = 0.43 \rho_s \frac{f_{yh}}{f'_c} \left(1 - 1.4 \frac{P}{f'_c A_g} \right) \quad (8.1b)$$

Where D_{ult} is the ultimate drift capacity, k_2 is the confinement efficiency parameter, P_0 is the nominal concentric compressive capacity of the column and P is the axial compressive force on the column.

Drift capacity of HSC columns in the current study is proposed in terms of the ratio and grade of HSS longitudinal reinforcements, concrete compressive strength, confinement level and the axial loading level. The approach is based on the calculation of drift capacities for HSC columns with different levels of confinement and axial loading. The computation of column drift capacities was performed using Diana 9.4.4. As shown in **Figure 8.8**, degradation in lateral load-displacement response is caused by the loss of moment capacity in the plastic hinge region as well as the $P - \Delta$ effect. Per common practice in literature, the lateral displacement

capacity of a column can be defined as a displacement value at which the lateral loading capacity of the column is equal to 80% of the maximum lateral load carrying capacity [B2, L1, L3, P6, S1].

Figure 8.8 Lateral load-displacement of RC columns

8.4.1 Effect of confining transverse reinforcement on drift capacity of columns

Previous studies [P5, R2] showed that different amounts of confinement reinforcement and material properties will not influence the drift capacity and lateral loading capacity as long as the value of $\rho_s f_{yh} / f'_c$ in an RC column remains constant. The influence of various grades of transverse steel reinforcement in RC columns was studied in this chapter using an FE approach. **Figure 8.9** summarizes the parametric study results illustrating the calculated drift capacity versus axial loading levels. It is observed that increasing the yield strength of column transverse reinforcement from $f_{yh} = 475$ to 1275 MPa will increase the drift capacity between 3 to 9% in the case of zero axial loading. However, as the axial loading is going higher to $0.6 f'_c A_g$, the influence of column transverse reinforcement deteriorates gradually. In the case of column axial loading of $0.6 f'_c A_g$, increasing the yield strength of column transverse reinforcement from $f_{yh} = 475$ to 1275 MPa will increase the drift capacity by less than 2%. In addition, it is

observed that in columns with higher ratios of longitudinal HSS reinforcing bars (ρ_{Hs}), the influence of utilizing higher grade transverse reinforcement is more significant up to the optimum ratio of $\rho'_t = 55\%$. In columns with $\rho'_t = 30\%$, utilizing transverse reinforcement with the yield strength of $f_{yh} = 1275$ MPa will increase the drift capacity up to 8% compared to utilizing normal-strength transverse reinforcement. This increase in drift capacity is around 10% and 6% in RC columns with $\rho'_t = 55\%$ and $\rho'_t = 100\%$, respectively. It is concluded that the most beneficial effect of utilizing HSS transverse reinforcement is obtained when the optimum ratio of $\rho'_t = 55\%$ is used.

Figure 8.9 shows the ultimate drift capacities versus volumetric confinement levels for the studied columns with different HSS reinforcement ratios. The value of $\rho_s f_{yh} / f'_c$ could be specified as a design parameter and linear trends have been shown for different axial loading levels. The general trend of lines indicates that the ultimate drift capacity will increase when a higher confinement level is provided. However, in a certain level of confinement, columns with lower axial loading levels of 0 and $0.1 f'_c A_g$ are observed to exhibit higher drift capacities. As the level of axial loading is increased, the slope of the drift capacity trend line is decreased. In order to propose an expression to relate the ultimate drift capacity of columns to the defined confinement level, analysis cases with zero axial loads are considered. A basic equation that expresses the RC columns drift capacity to the confinement level is presented as:

$$D_{P=0} = 0.4 \rho_s \frac{f_{yh}}{f'_c} \quad (8.2)$$

Where $D_{P=0}$ is the ultimate drift capacity of a column with zero axial loading, ρ_s is the volumetric ratio of transverse reinforcement, f_{yh} is the yield strength of transverse reinforcement and f'_c is the concrete compressive strength.

Figure 8.9 Estimation of the drift capacity of RC columns

8.4.2 Effect of axial loading level on drift capacity of columns

As shown in **Figure 8.10**, the presence of axial loading has a detrimental effect on drift capacity of columns and the drift capacity decreases as the axial loading level increases from 0 to 0.6. In columns with Grade 60 concrete, drift capacity decreases between 48 to 62% when the axial loading level is increased from 0 to 0.6. The decrease in drift capacity is between 42 to 60% and 39 to 58% in RC columns with Grade 80 and Grade 100 concrete, respectively. It was concluded that the effect of axial loading level on deterioration of column drift capacity is more significant in lower concrete grades.

In order to find a comprehensive expression of ultimate drift capacity, **Equation 8.2** is modified for different axial loading levels. Assuming that any increase in the column axial loading will reduce the ultimate drift capacity linearly, a reduction factor should be applied to **Equation 8.2**. As a solution to find the reduction factor, drift values for the cases with axial loading are divided into the drift value of the similar models with zero axial loads. This parameter is specified as the normalized drift value. Drift values in the case of zero axial loading are obtained using **Equation 8.2**. Performing a regression analysis on drift capacity values for different axial loading levels ranging from $0.1 f'_c A_g$ to $0.6 f'_c A_g$, the following expression can be presented for the ultimate drift capacity:

$$\text{Reduction factor} = 1.5 \left(1 - 0.9 \frac{P}{f'_c A_g} \right) \quad (8.3)$$

Combining **Equations 8.2** and **8.3**, the following expression can be obtained to relate the drift capacity of columns to confinement ratio and axial load level factors:

$$D_{ult} = 0.6 \rho_s \frac{f_{yh}}{f'_c} \left(1 - 0.9 \frac{P}{f'_c A_g} \right) \quad (8.4)$$

8.4.3 Effect of reinforcement yield strength on drift capacity of columns

The effect of the yield strength of longitudinal reinforcing bars on drift capacity of RC columns is investigated in this section. FE models with various grades of

longitudinal bars were analyzed and the result is illustrated in **Figure 8.10**. To find the relation between the drift capacity and the yield strength and ratio of longitudinal HSS bars, FE parametric studies were performed for steel grades of 700, 800 and 950 MPa.

Increasing the yield strength of longitudinal bars from 700 to 950 MPa will result in a 10% decrease in drift capacity of columns. It was noted that the grade of longitudinal steel reinforcement was more influential in the case of moderate axial loading between $0.2 f'_c A_g$ to $0.4 f'_c A_g$. In extreme values of axial loading of 0 and $0.6 f'_c A_g$, altering the grade of longitudinal steel reinforcement has a minor effect on drift capacity of columns. In addition, it is observed that the influence of utilizing longitudinal HSS bars is more significant in RC columns with higher concrete grades.

The general trend shows that increasing the ratio of longitudinal HSS bars from $\rho'_t = 30\%$ to $\rho'_t = 55\%$ will increase the drift capacity between 4 to 17%. However, any further increase of ρ'_t from 55 to 100% will decrease the drift capacity by 3 to 10%. It is concluded that among all the possible combinations of mixed grade longitudinal reinforcements in RC column sections, columns with longitudinal HSS bar ratio of $\rho'_t = 55\%$ show higher drift capacities.

Considering the ratio and strength of longitudinal reinforcements in the conducted parametric study, **Equation 8.4** would be rearranged as follow:

$$D_{ult} = 0.6 \rho_s \frac{f_{yh}}{f'_c} \left(1 - 0.9 \frac{P}{f'_c A_g} \right) \left(\frac{\rho_t f_y}{\rho'_t f_{yHs}} \right) \quad (8.5)$$

Where ρ_t is the total reinforcement ratio in the column section, f_y is the yield strength of ordinary steel in the section, ρ'_t is the ratio of longitudinal HSS reinforcements area divided by the total longitudinal reinforcement area in column section, and f_{yHs} is the yield strength of the HSS reinforcement.

Figure 8.10 Ultimate drift capacity versus axial load level for RC columns

8.5 Drift-Based Design Charts for HSC Columns

For application, it is a common practice to consider the cases which present more conservative drift expressions. Hence, the analysis results of the columns with shear span ratio of 2.5, which produce lower drift capacities, are considered in proposing the design formulation. Allowable story drifts for most of concrete frames are determined to be around 2 to 2.5% according to current building codes [I1, U1]. While the proposed equation can be used for different drift ratios, the allowable ultimate drift ratio could be specified to be 4% for HSC frames in high seismic regions. Substituting this allowable drift ratio in **Equation 8.5** and solving for the HSS reinforcement ratio, a design equation can be developed as follows:

$$\frac{\rho'_t f_{yHs}}{\rho_t f_y} = 14.1 \rho_s \frac{f_{yh}}{f'_c} \left(1 - 0.9 \frac{P}{f'_c A_g} \right) \quad (8.6)$$

Similar design equations are proposed for higher ultimate drift ratios as well. Design equations are plotted to qualify the effect of different design parameters on the ultimate drift capacity of columns. **Figure 8.11** shows the interaction between

$\rho_s \frac{f_{yh}}{f'_c}$ and $\frac{\rho'_t f_{yHs}}{\rho_t f_y}$ for certain ultimate drift ratios. The graphs are plotted for

different axial loading levels and it was observed that the line gradient for higher axial loading levels is smaller. In order to summarize the proposed design expressions, several design charts have been developed in this study. **Figure 8.12** shows design charts for certain ultimate drift values ranging from 4 to 7%. Axial loading levels vary from 0 to $0.6 f'_c A_g$ in the vertical axis and the minimum value of

$\rho_s \frac{f_{yh}}{f'_c}$ defined by ACI 318-11 [A1] is shown in charts as a vertical line. An

example is presented here in order to illustrate the design procedure using the proposed design charts. Assume that an RC column which is under an axial loading of $0.35 f'_c A_g$ is required to be designed in a way that it could undergo a drift capacity value of $D_{ult} = 5\%$. Assume that known parameters of transverse reinforcement ratio (ρ_s), transverse reinforcement yield strength (f_{yh}) and

concrete compressive strength (f_c') will lead to the design value of $\rho_s \frac{f_{yh}}{f_c'} = 0.2$.

Specifying the level of axial loading $\left(\frac{P}{f_c' A_g} \right)$ and confinement ratio $\left(\rho_s \frac{f_{yh}}{f_c'} \right)$ in the

design graph of **Figure 8.11** for the case of $D_{ult} = 5\%$, the intersection point will

drop on the curve of $\frac{\rho_t' f_{yHs}}{\rho_t f_y} = 1.5$. As a result, the ratio of longitudinal HSS

reinforcement in the column section (ρ_t') and its yield strength (f_{yHs}) can easily be found.

Figure 8.11 Interaction between longitudinal reinforcement yield strength and the confinement level

Figure 8.12 Drift-based design charts for RC columns

8.6 Conclusions

A study was carried out to investigate the seismic performance of HSC columns with HSS reinforcement. To evaluate the design method of such structural members, comprehensive finite element analyses and analytical studies were performed in order to present a drift-based design formulation. The following conclusions are drawn based on test results and theoretical considerations:

1. Utilizing arrangements of HSS reinforcement can significantly improve the flexural behavior of HSC columns under severe earthquake loadings.
2. Drift capacities were consistently higher for columns with higher shear span-depth ratios L/h . Drift-based design formulation was developed for columns with shear span ratio of 2.5, which produces lower estimates of the drift capacity.
3. Drift capacity of columns decreases as the axial loading value increases from 0 to $0.6 f'_c A_g$. However, the effect of axial loading level on drift capacity deterioration is more significant in columns with lower concrete grades.
4. It was observed that the grade of longitudinal steel reinforcement was more influential in the case of moderate axial loading levels between $0.2 f'_c A_g$ to $0.4 f'_c A_g$.
5. The HSS ratio of $\rho'_t = 55\%$ in columns can be suggested as an optimum design parameter in drift-based design of RC columns utilizing HSS longitudinal reinforcement.
6. Design equations were proposed and design charts were plotted to qualify the effect of different design parameters on the ultimate drift capacity.

CHAPTER 9

ANALYTICAL STUDIES OF BEAM-COLUMN JOINTS USING STRUT-AND-TIE MODELLING

9.1 Introduction

The concrete diagonal strut mechanism and parallel angle truss model proposed by Park and Paulay [P2] has been a method to obtain seismic design equations for the joints of moment resisting frames. Restrepo *et al.* [R4] concluded that the concrete diagonal strut and parallel truss model is unable to satisfy the equilibrium of internal forces all the time and they proposed a variable-angle truss model. In the current study, the analytical model proposed by Lin [L5] was used to analyze the performance of the tested specimens. The model is based on the lower bound theorem of plasticity and the evaluation of the flow of internal forces is performed using a strut-and-tie model. In order to evaluate the internal force flow within the joint panel, bond forces along the longitudinal beam bars were allocated to the nodes where beam bars met the column interior bars. The bond forces were calculated based on the proposed equation in this chapter. In addition, the longitudinal column bar forces have been calculated from a section moment-curvature analysis. However, longitudinal beam bar forces were obtained based on the assumption of plastic hinge formation in beams at joint faces. The forces in horizontal joint reinforcement have been replaced with an equivalent uniformly distributed stress block which is acting at vertical faces of the joint panel. The predicted cracking pattern of tested specimens was compared to experimental observations.

9.2 Strut-and-Tie Model Proposed by Lin (2000)

9.2.1 General

In the model proposed by Lin [L5], the longitudinal beam top and bottom reinforcement is called “lumped reinforcement”. The first step in the analysis procedure is to evaluate the forces in beams and columns adjacent to the joint panel. Based on the current seismic design philosophy of RC frames, columns remain

elastic while plastic hinges form in beams framing into joints. As a result, the transmitted forces from columns to the joint can be reasonably estimated using moment-curvature section analysis. However, the tensile forces in longitudinal beam bars at column faces are calculated as the nominal yield force of the steel multiplied by an over-strength factor of 1.25 which is suggested by NZS 3101-06 [N3]. On the other hand, the compressive force at the beam compression zone at the column face consists of two components; one due to the concrete and the other one due to the beam compressive longitudinal reinforcement. The centroid of the concrete compressive stress is quite close to the level of compressive longitudinal reinforcement when the beam flexural capacity is reached. Hence, the beam concrete compressive force is considered to be single force acting at the compressive reinforcement level. It is to be noted that the magnitude of the beam concrete compressive force depends on the bond strength of beam bars passing through the joint region. The higher developed bond force along longitudinal beam bars will result in lower beam concrete compressive force.

9.2.2 Modeling assumptions

Some basic assumptions have been made in strut-and-tie modeling of all the beam-column joints in the current study. These assumptions can be listed as follows:

- (a) The concrete in the joint panel is assumed to be cracked.
- (b) Plastic hinges are only formed in all beams framing into the joint.
- (c) The columns framing into the joint remain elastic.
- (d) The concrete compressive force in beams which is resulted from flexure is acting at the level of the beam compressive reinforcement.
- (e) The column shear force at the joint face is distributed only within the region of the column concrete compressive stress block.
- (f) The beam shear force at the column face enters the beam-column joint as a concentrated force at a location close to the beam compressive steel.
- (g) Bond forces along the longitudinal beam bars passing through the joint region cannot exceed those computed from a given bond stress law. An explanation to the proposed bond stress law is presented in Section 9.3.

- (h) The confining effect of horizontal joint reinforcement is assumed to be uniformly distributed along two sides of the joint panel.
- (i) An equivalent rectangular stress block replaces the column concrete compressive stress block at the face of the joint with the centroid coinciding with that of the original triangular profile.

9.2.3 Strut-and-tie solution

The typical strut-and-tie model of an interior beam-column joint proposed by Lin [L5] is shown in **Figure 9.1**. In each joint, equilibrium must be insured between the input forces, struts and reinforcement or ties. The diagonal compression field in the joint is modeled with several diagonal struts. In addition, bond forces in the longitudinal beam bars are allocated at the nodes where beam bars meet column interior bars. The allocated bond forces are calculated based on the method which will be explained in Section 9.3. The remaining bond force is allocated to other struts to satisfy equilibrium.

The angle of struts can be determined based on the level of the nodes on column exterior bars. The lateral pressure of horizontal joint reinforcement is also applied to the nodes on column exterior bars. The vertical component of the compressive force taken by the struts can be calculated when the angle of the struts are determined. It should be noted that the calculated tensile force in column interior bars cannot exceed the steel yield strength. In addition, interior nodes where the struts crossing the column interior bars are usually required to achieve equilibrium. A few iterations are required in this step.

The average uniaxial stress in the center of the joint panel $f_{c,s}$ is defined as follows:

$$f_{c,s} = \frac{s_c}{w_s b_j} \quad (9.1)$$

Where s_c is the force in central strut, w_s is the half of distance between the struts at either side of the strut and b_j is the joint width.

Figure 9.1 The analysis of beam-column joints using strut-and-tie models [L5]

The uniaxial compressive stress in the joint center does not represent the maximum uniaxial stress in the joint which occurs in the region close to the column and beam compression zones. However, Lin [L5] has stated that the uniaxial stress in the joint center could be used as a failure index as experimental work has shown that joint failure occurs by crushing of the concrete at the joint center.

The uniaxial compressive stress of the diagonal strut which is calculated as the method explained above is sensitive to the number of struts used in the joint. Hence, the number of struts in the current strut-and-tie modeling is kept as relatively constant. As a result, the effect of strut numbers on the calculated values of $f_{c,s}$ can be reduced and the described methodology can be used to measure the influence of other parameters.

The strut-and-tie model proposed by Lin [L5] is adopted in this study in order to model the force flow in beam-column joints. This model is statically determinate

and only equilibrium condition is used to achieve the solution. Strain compatibility or constitutive law consideration is not required in this modeling solution. However, several solutions may be found to satisfy admissible equilibrium but a solution with the lowest $f_{c,s}$ will be selected in order to conform to the lower bound theorem of plasticity.

9.3 Bond Strength Modeling

In the profile of bond distribution model proposed by Restrepo *et al.* [R4], the bond stress distributes linearly from zero to its maximum value across the column's effective length. However, severe pinching behavior was observed in the hysteresis response of the test specimens in the current study which is due to the significant bond-slip of beam bars. In the present study, following the studies of Lowes and Altoontash [L6] and Hwang *et al.* [H4], a simplified bond-strength model is adopted as shown in **Figure 9.3**. In this model, the bond stresses were simplified as uniform effective stresses of τ_e and τ_u before and after reinforcing bar yielding. The value of the friction bond strength τ_u can be evaluated using the test geometry parameters as follows.

$$\tau_u = \frac{2P_0H}{h_s \left(\sum_{i=1}^{n_b} \pi d_{bi} h_c \right) (h_c / L_1 + 2)} \quad (9.2)$$

Where P_0 is the lateral applied loading at the origin in the loading cycle in which the complete bond failure of the specimen occurs. H is the total height of the column, L is the length of the beam, h_c is the column depth, h_s is the distance between top and bottom flexural bars in beam section and L_1 is the distance between beam supports and the column face. The calculated values of τ_u for the test specimens are presented in **Table 9.1**.

Discrete potential bond forces

$$U_a = 2.2\sqrt{x_t f'_c} \frac{(X_a)^2}{2d_c - h_c} \frac{s_0}{2}$$

$$U_b = 2.2\sqrt{x_t f'_c} \left(\frac{X_a + X_b}{2d_c - h_c} \right)^2 \frac{s_0}{2} - U_a$$

Figure 9.2 The bond stress law of beam bars passing through the joint proposed by Restrepo [R4]

Figure 9.4 illustrates the bond stress distribution along the beam flexural rebars in beam-column joints subjected to cyclic loadings. As described before, the bond stress distribution has been simplified using the bearing bond strength (τ_e) and the friction bond strength (τ_u). It is assumed that as the inelastic deformation of the beam bars increases, yielding of reinforcing bars further penetrates into the joint and the bond strength is reduced respectively. **Figure 9.4(b)** shows the case of complete bond failure in which the bond resistance is further degraded. In this case, the friction bond strength (τ_u) is assumed to be uniformly distributed along the beam flexural bars over the entire joint depth. In the beam-column joints tested in this study that significant bond slip was observed, severe pinching effect occurred in the hysteresis response of the specimens. Hence, the loading and unloading stiffness decreased to zero at the origin of the lateral load versus displacement relationships.

Figures 9.5 and 9.6 show the lateral loads P_0 of the test specimens which were used to evaluate the friction bond strength in the current study. The P_0 values used in the bond strength calculations were the average of the positive and negative loadings at the last loading cycle.

The variation of the friction bond strength τ_u versus design parameters are expressed in **Figure 9.7**. The data points summarize the friction bond strengths τ_u that were calculated from the current test results and the specimens tested by Li and Leong [L2]. All the test results are within the τ_u range defined by Lowes and Altoontash [L6]. It was observed that the concrete compressive strength, joint shear strength ratio, bar spacing ratio of the beam flexural bars and the axial loading level are among the design parameters which have a noticeable effect on the value of τ_u . The correlation of τ_u with the concrete compressive strength is shown in **Figure 9.7(a)**. However, a large deviation is observed.

Based on the analysis results, the friction bond strength is defined as a function of design parameters as follows.

$$\frac{\tau_u}{\sqrt{f'_c}} = 0.06 + 0.04 \left(\frac{v_{sh} \cdot s}{v_u \cdot d_b} \right) \left(\frac{f'_c A_g}{f'_c A_g + P} \right) \quad (\text{MPa}) \quad (9.3)$$

Where v_{sh}/v_u is the ratio of provided shear strength by joint hoops and s is the clear bar spacing.

Figure 9.3 Simplified bond strength

Figure 9.4 Bond-stress distribution of beam reinforcing bars in joints under cyclic loading [H4]

Figure 9.5 Determination of P_0 in joints with complete bond failure tested in this study

Figure 9.6 Determination of P_0 in joints with complete bond failure in Li and Leong specimens [L2]

Figure 9.7 Variation of friction bond strength in terms of design parameters

9.4 Strut-and-Tie Modeling of Test Specimens

9.4.1 Analysis results for interior joints

Figures 9.8 through 9.10 show the possible strut-and-tie models for the test specimens and the internal force flow due to seismic actions. The calculation steps and modeling procedure is discussed in Appendix IV. Positive and negative loading cracks which are determined based on the force flow mechanism correlates to the observed experimental cracking. The values of the main strut compressive force C_s , the width of the strut w_s , the uniaxial compressive stress of the diagonal strut at the joint center $f_{c,s}$, theoretical and experimental joint shear stress v_{jh} and the ratio of $f_{c,s}/v_{jh}$ is summarized in Table 9.1 for all of the specimens. It is observed that the maximum value of $f_{c,s}/v_{jh}$ belongs to Specimen IH60 with T19 beam bars. Comparing IN80 and IN100 with their similar specimens of IH80 and IH100 with Grade 700 bars clarifies that utilizing HSS beam reinforcements increases the joint shear stress and the value of $f_{c,s}/v_{jh}$ which is compatible with experimental observations. In addition, it is observed that the uniaxial compressive stress in IN80 joint with $v_{sh}/v_{jh} = 1$ is considerably smaller than that in IH80 with $v_{sh}/v_{jh} = 0.84$. This observation clearly shows the beneficial effect of providing adequate joint transverse reinforcements in order to reduce the joint shear stress.

Lin [L5] concluded that increasing $P/f'_c A_g$ from 0.1 to 0.3 or any increase in the value of v_{sh}/v_{jh} will reduce the $f_{c,s}/v_{jh}$ ratio. In the current study, the analysis results of the IH80 and IH60 were compared to IH80A and IH60A with the presence of axial loadings. It was observed that the uniaxial compressive stress slightly decreased when the axial load level increased from zero to $0.3 f'_c A_g$.

9.4.2 Analysis results for exterior joints

Figures 9.11 and 9.12 show the possible strut-and-tie models for the test specimens and the internal force flow due to seismic actions. Positive and negative loading cracks which are determined based on the force flow mechanism correlates to the observed experimental cracking. Joint diagonal shear cracking were observed in

early-stage loading cycles in Specimens EN80, EH80 and EH60. However, joint cracks did not widen through the test and all the specimens failed due to beam plastic hinge formation. As shown in **Figure 9.13**, positive and negative loading cracks are determined based on the force flow mechanism and they are compared to the observed cracking of test specimens. Due to the joint confinement ratio of $V_{sh}/V_{jh} = 1.0$ in all specimens, it is observed that the horizontal forces caused by the participation of joint hoops are sufficient to balance the horizontal joint shear and bond stresses.

The values of the main strut compressive force C_s , the width of the strut w_s , the uniaxial compressive stress of the diagonal strut at the joint center $f_{c,s}$, theoretical and experimental joint shear stress v_{jh} and the ratio of $f_{c,s}/v_{jh}$ is summarized in **Table 9.1** for all of the specimens. It is observed that the maximum value of $f_{c,s}/v_{jh}$ belongs to Specimen EH60 with T19 beam bars. Comparing EN80 with its similar specimen of EH80 with Grade 700 bars clarifies that utilizing HSS beam reinforcements increases the joint shear stress and the uniaxial compressive stress $f_{c,s}$ which is compatible with experimental observations. However, the value of $f_{c,s}/v_{jh}$ did not alter significantly when utilizing HSS reinforcements in EH80 compared to EN80.

Table 9.1 Summary of strut-and-tie modeling results

Specimen	IN80	IH80	IN100	IH100	IH60	EN80	EH80	EH60
$C_s (KN)$	410	637	294	602	633	353.5	359.5	302
$w_s (mm)$	68.4	73.0	76.3	73.7	87.5	84.2	63.5	54.3
$f_{c,s} (MPa)$	23.9	34.9	15.4	32.7	28.9	16.8	22.6	22.2
$v_{jh} (MPa)$	5.8	7.6	5.8	7.6	7.6	3.8	5.4	3.8
v_{sh}/v_{jh}	1.0	0.8	1.0	1.0	0.9	1.0	1.0	1.0
$v_{jh,experimental} (MPa)$	6.0	5.4	4.1	5.5	6.2	4.0	4.4	3.3
$f_{c,s}/v_{jh}$	4.1	4.6	3.2	4.0	3.8	4.4	4.2	5.8
τ_u	3.8	3.7	3.2	4.8	0.9	-	-	-

Figure 9.8 Strut-and-tie analysis of Specimens IN80 and IH80

Figure 9.10 Strut-and-tie analysis of Specimens IH60 and IH60A

Figure 9.11 Strut-and-tie analysis of Specimens EN80 and EH80

Figure 9.12 Strut-and-tie analysis of Specimens EH60

9.5 Verification of Strut-and-Tie Models using the Cracking Pattern of Test Specimens

In order to verify the proposed bond-strength model, the cracking pattern predicted by the strut-and-tie modeling is compared with the experimental observations. Joint diagonal shear cracking were observed in early-stage loading cycles in Specimens IN80, IH80, IN100, IH100 and IH60. However, joint cracks did not widen through the test and all the specimens failed due to beam plastic hinge formation. **Figure 9.13** shows the predicted cracking pattern of internal force flow in specimens due to seismic actions. Positive and negative loading cracks are determined based on the force flow mechanism and they are compared to the observed cracking pattern of test specimens. As observed, the internal compression forces in the beam and column compression zones use the corner-to-corner path to balance each other. Hence, the diagonal cracks occurred along the major force paths. In all of the specimens, joint transverse reinforcements are participating in the load carrying mechanism and a part of the internal compression forces are required to balance the transverse forces. As a result, the paths from the compression sides of the beams and columns to the middle parts of the beams are implemented.

As observed in the specimens with T19 longitudinal beam bars, the angles of the force paths are not as steep as other specimens. The reason is due to the smaller bond forces that were imposed on the interior nodes of the column. As explained in **Figure 9.4**, the bond strength of Specimen IH60 will be degraded after the bond-slip of the reinforcing bars. The reduced bond force will reduce the horizontal force in diagonal struts, which will alter the strut angle. In general, an agreement was observed between the predicted and observed cracking pattern of the specimens.

Figure 9.13 Predicted cracking pattern of test specimens

9.6 Conclusions

Analytical study on the behavior of interior and exterior joints was conducted in this chapter. The study focused on the bond mechanism inside the joints and a calibrated bond-slip law was incorporated in the strut-and-tie model proposed by Lin [L5]. The following conclusions are drawn in this chapter:

1. It was observed that the bond modeling has a great effect on the force flow mechanism and cracking pattern of the joints. Various bond stress distributions and bond-slip laws would result in a different force flow mechanism inside the joints.
2. A bond distribution model was proposed based on the observed pinching behavior of tested specimens in this study. The model considers the effect of the concrete compressive strength, diameter of longitudinal beam bars, the spacing of the reinforcing bars, shear strength of the joint hoops, and the effect of the axial loading ratio.
3. To assess the proposed model, the force flow mechanism in joints was compared between the experimental results and the STM prediction. The comparison demonstrated very good agreement in the shear stress in joint center and the cracking pattern.

CHAPTER 10

SUMMARY AND CONCLUSIONS

10.1 Conclusions

Experimental, numerical, and analytical investigations were conducted in this study to investigate the seismic behavior of high-strength concrete (HSC) members and frames reinforced with high-strength steel (HSS).

The seismic performance of HSC beam-column joints with high-yield strength steel reinforcements was evaluated through the experimental and analytical approach in this study. The aim of the study was to investigate whether the ACI 318-11 [A1] code requirements of the moment strength ratio, the shear strength, and the reinforcement anchorage could be extended to the design of beam-column joints constructed using high-strength materials. The increased bond-slip of reinforcing bars, decreased hysteretic energy dissipation, and lower shear strength of the joints are the key issues in designing beam-column joints utilizing HSS reinforcements. The γ values of joint shear strength of the specimens were between $0.4\sqrt{f'_c}$ and $0.9\sqrt{f'_c}$ which are considerably less than the value of $1.245\sqrt{f'_c}$ suggested by ACI 352-02 [A3]. However, the performance of test specimens were found to be satisfactory up to a drift ratio of 4.0%. However, those specimens with larger beam bar diameter of T19 did not reach their theoretical lateral loading capacities. Strain profiles of reinforcements showed that yielding happened along the beam bars at the column face during the final loading cycles of the test in all of the specimens, which is the desirable failure mechanism. Axial compression loading increased the energy dissipation significantly and utilizing HSS reinforcements resulted in a smaller energy dissipation capacity. Utilizing longitudinal beam reinforcements of higher grade caused a slight decrease in bond strength in the joint region. A modified friction bond strength model was proposed and verified using the test results in this study to further investigate the effect of utilizing HSS reinforcing bars.

Based on a literature study, experimental program, and finite-element investigations, it was concluded that the current code provisions could be used for the design of high-strength members. However, several modifications should be suggested to prevent premature bond-slip in interior beam-column joints. Some limitations on the beam-bar diameter to the column-depth ratio d_b/h_c were suggested in this study and a comparison was made between the proposed equation and the values suggested by ACI 318-11 [A1] and NZS 3101-06 [N3] codes. Assuming the column axial loading of $0.3f'_cA_g$ and the concrete compressive strength of 80 MPa, the required joint depth is suggested as $h_c = 20d_b$ for Grade 500 reinforcement ($f_y = 500\text{MPa}$), and $h_c = 32d_b$ for Grade 700 reinforcement ($f_y = 700\text{MPa}$). However, increasing the concrete grade to $f'_c = 100\text{MPa}$ could reduce the required joint depth to $h_c = 20d_b$, which is identical to ACI 318-11 [A1] recommendation.

For exterior beam-column joints, experimental evidence, and analytical models conclusively show that the required development length is increased significantly when the yield strength of reinforcing bars is increased. To overcome this issue, using HSC is a key solution. Utilizing HSC coupled with HSS reinforcement will result in a considerable reduction in the required development length. In addition, the beneficial effect of column axial loading, which is because of its confining effect on the joint, is obvious. As a general conclusion, the required development length (l_{dh}/d_b), in the case of using HSS reinforcement, could be controlled if the effect of column axial loading and HSC use is incorporated. Hence, a novel expression was proposed to determine the development length of the longitudinal beam reinforcement passing through exterior joints.

In addition, the correlation between the ultimate drift capacity of columns and design parameters was investigated and design charts were presented. The effect of concrete grade, the yield strength of steel bars, and the axial loading level was incorporated in design equations.

The conclusions are categorized in the sections that follow.

10.1.1 The influence of variables based on the literature review

A total of 92 beam-column joints from the test database were collected to study the influence of parameters such as the concrete compressive strength, the bond index, the column axial loading, the ratio of joint hoops, and the yield strength of steel reinforcement in beams and columns. The following conclusions were drawn:

1. In general, the following parameters have significant effects on seismic behavior of RC beam-column joints: the concrete compressive strength, the yield strength of beam reinforcing bars, the bond index, and the ratio of joint hoops. In addition, the column axial compression loading affects the drift capacity of the beam-column joints.
2. The influence of concrete compressive strength on ultimate capacity of joints is less significant as compared to other parameters. In the current study, the joint shear strength was normalized to the concrete compressive strength to observe the influence of other control parameters.
3. Column axial compressive loading has no direct effect on the maximum joint shear strength in general. However, it affects the joint shear deformation and may cause different deterioration behavior after the peak column shear strength is reached.
4. Longitudinal reinforcement passing through the joint region will help in resisting the joint shear stress. However, excessive longitudinal reinforcement may change the failure mechanism from the beam hinging to the joint shear.
5. The bond index (μ) can be an appropriate indication to estimate the bond condition of the reinforcement in the joint. It is recommended that the application of the bond index in designing the development length of the reinforcement should be accompanied by checking the longitudinal reinforcement ratio.

10.1.2 Interior beam-column joints

The seismic performance of HSC beam-column joints with HSS reinforcement was evaluated through the experimental approach in this study. The following conclusions were found based on the results and further conducted investigations:

1. The performance of test specimens was found to be satisfactory up to a DR of 4.0%. Degradation in strength and stiffness occurred in Specimens IN80 and IN100 at a DR of 4.0% while there was no strength degradation in IH80 and IH100 with HSS reinforcements. Specimens IH60 and IH60A with a larger beam bar diameter of T19 did not reach their theoretical lateral loading capacities.
2. Strain profiles of reinforcements show that yielding happened along the beam bars at the column face during the final loading cycles of the test in all the specimens. This is compatible with the measured curvature distribution along the beams.
3. The stiffness degradation in Specimens IH80 and IH100 was observed to be less than that in IN80 and IN100 because of the use of HSS reinforcements. The axial compression loading was crucial to increasing the stiffness degradation of specimens and the role of concrete grade observed to be negligible.
4. Axial compression loadings increased the energy dissipation capacities significantly and utilizing HSS reinforcements resulted in a smaller energy dissipation capacity. Specimen IH80A with axial loading showed the largest energy dissipation capacity while IH60 with a larger beam bar diameter of T19 showed the smallest energy dissipation capacity.
5. The maximum nominal joint shear stress in the specimens was between $0.04 f'_c$ to $0.10 f'_c$, which is less than the limit of $0.25 f'_c$ suggested by NZS 3101-06 [N3]. Therefore, the joint failure mechanism was not expected in the specimens. The γ values of joint shear strength of the specimens were between $0.41 \sqrt{f'_c}$ and $0.85 \sqrt{f'_c}$, which are considerably less than the value of $1.245 \sqrt{f'_c}$ suggested by ACI 352-02 [A3].
6. Utilizing higher grade longitudinal beam reinforcements will cause a slight decrease in bond strength in the joint region. However, it was observed that using HSC coupled with HSS reinforcements improves the bond condition of the specimens. Column axial loads proved to have enhancing effects on the bond strength as well.

7. Analytical studies showed that using equivalent beam bars with higher yield strengths will reduce the energy dissipation capacity and increases the stiffness degradation. In addition, the required joint depth to prevent bond-slip is larger. However, utilizing HSC or considering the column compression loading would improve the joint condition.

10.1.3 Exterior beam-column joints

The seismic performance of HSC exterior beam-column joints with HSS reinforcement was evaluated through the experimental approach in this study. The following conclusions were determined based on the results and further conducted investigations:

1. The performance of test specimens was found to be satisfactory up to a DR of 4.0%. Degradation in strength and stiffness occurred in Specimens EN80 and EN60 at a DR of 2.5% and 4.0% while there was no strength degradation in other specimens. All the test specimens reached their theoretical lateral loading capacities in both positive and negative loading directions.
2. Strain profiles of reinforcements show that yielding happened along the beam bars at the column face during the final loading cycles of the test in all of the specimens. This corroborates the measured curvature distribution along the beams.
3. The stiffness of Specimen EH80 was observed to be higher than that of EN80 between 5% and 17% during the test, which was because of the use of HSS reinforcement. However, the stiffness degradation in EH80 was around 4% higher than that in EN80 at the end of the test. The axial compression loading was also crucial to increasing the stiffness of specimens.
4. Axial compression loadings increased the energy dissipation significantly and utilizing HSS reinforcements resulted in a smaller energy dissipation capacity. Specimen EH80A with axial loading showed the largest energy dissipation capacity while EH60 with a larger beam bar diameter of T19 showed the smallest energy dissipation capacity.

5. The maximum nominal joint shear stress in the specimens was around $0.05 f'_c$, which is less than the limit of $0.25 f'_c$ as suggested by NZS 3101-06 [N3]. Hence, the joint failure mechanism was not expected in the specimens. The γ values of joint shear strength of the specimens were between $0.37 \sqrt{f'_c}$ and $0.50 \sqrt{f'_c}$, which are considerably less than the value of $0.996 \sqrt{f'_c}$ as suggested by ACI 352-02 [A3].
6. Utilizing higher grade longitudinal beam reinforcements will cause a slight decrease in bond strength in the joint region. However, it was observed that using HSC coupled with HSS reinforcements improves the bond condition of the specimens. Column axial loads proved to have enhancing effects on the bond strength as well.
7. Analytical studies showed that using equivalent beam bars but with higher yield strengths reduces the energy dissipation capacity, stiffness values, and the stiffness degradation. In addition, the required development length to prevent bond-slip is to be larger. However, utilizing HSC or taking into consideration the column compression loading would improve the joint condition.
8. The required development length of the longitudinal beam bars (l_{dh}/d_b) increases significantly when the reinforcement with high yield strength is used. Utilizing HSC coupled with HSS reinforcement as well as the incorporation of the effect of column axial loading are the key solutions to overcome this issue.

10.1.4 Mixed-grade longitudinal steel reinforcement in RC columns

1. Utilizing HSS reinforcement by the ratio of 30%, 50%, and 100% in an RC column section can increase the lateral loading capacity by 18%, 26%, and 55% on average, respectively.
2. Maximum lateral loading capacity increases between 60% and 85%, when the column shear span ratio decreases from 5.0 to 2.5. In addition, drift capacities were consistently higher for columns with higher shear span-depth ratios L/h . Drift-based design formulation was developed for columns with a shear span ratio of 2.5, which produces lower estimates of the drift capacity.

3. The drift capacity of columns decreases as the axial load value increases from 0 to $0.6 f'_c A_g$. However, the effect of axial load level on drift capacity deterioration is more significant in columns with lower concrete grades.
4. The influence of the yield strength of reinforcing bars on the drift capacity of HSC columns is less significant as compared to the influence of other design parameters. However, it was noted that the grade of longitudinal steel reinforcement was more influential in the case of moderate axial load levels between $0.2 f'_c A_g$ to $0.4 f'_c A_g$. In addition, it was observed that the effect of HSS longitudinal reinforcement is more significant in RC columns with higher concrete grades.
5. Among all the possible combinations of mixed grade longitudinal reinforcement in RC column sections, the columns with an HSS bar ratio of $\rho'_t = 55\%$ show higher drift capacities. This value can be suggested as an optimum design parameter in drift-based design of HSC columns utilizing HSS longitudinal reinforcement.
6. The influence of various grades of transverse steel reinforcement in HSC columns was studied. It was observed that increasing the yield strength of column transverse reinforcement from $f_{yh} = 475 \text{ MPa}$ to $f_{yh} = 1275 \text{ MPa}$ increases the drift capacity between 3% and 9% in the case of zero axial load level. However, as the axial load level increases to $0.6 f'_c A_g$, the influence of column transverse reinforcements deteriorates gradually.
7. In columns with higher ratios of HSS longitudinal reinforcement ρ'_t , the influence of utilizing higher grade of transverse reinforcement is more significant up to the optimum ratio of $\rho'_t = 55\%$.

10.2 Suggested Future Research

The research data on HSC frames and members with HSS reinforcement is inadequate in literature and excessive experimental studies are required to provide a better understanding of the seismic behavior of these structural members. Hence, the following topics are recommended for further research:

1. The applicability of current design approaches and the proposed methods in this study are subjected to the assumptions made on material properties. However, the mechanical properties of beam reinforcing bars may affect the spread of plasticity within the plastic hinge regions. As a result, future research should focus on the stress-strain curve of HSS reinforcement and its effect on the sectional strength and deformation capacity of the members.
2. The axial loading value in the current study was set to be $0.3 f'_c A_g$. The beneficial effect of axial loading and the effect of its various values were investigated using the finite-element approach. Hence, future experimental studies are expected to be conducted under various levels of column compressive axial loadings.
3. ACI 318-11 [A1] does not contain any recommendations to prevent the splitting bond failure in moment frames. Existing tests have not addressed splitting bond failure and future research could investigate the potential for this failure mechanism.
4. The current lower limit on concrete compressive strength specified in concrete structures standards is set based on the use of normal-strength reinforcing bars. Future research should focus on determining the lower limit of f'_c when HSS bars are utilized.
5. Further studies are required to investigate the seismic behavior of 3D joints. The effect of floor slabs on determining the required joint depths should be investigated. Reduction in the required transverse reinforcement ratio in the joint region due to the confining effect of floor slabs, transverse beams, and column axial loading in 3D joints is to be investigated.
6. The geometry and dimensions of the members were identical for all of the test specimens in the current study. However, the effect of using deep beams or deep columns on the force flow mechanism in the joint panel needs to be further studied. Further studies on roof joints are necessary due to the lack of data on the seismic behavior of such joints.
7. Further improvements are required in the field of finite-element analysis of RC members to better match the experimental results when high-strength material is utilized. Contemporary models of concrete elements, high-strength

- reinforcement elements, constitutive relationships, and the bond-slip relationship of steel reinforcement are to be developed.
8. Current design provisions for establishing the ratio of beam-bar diameter to column-depth in interior joints, and the development length of beam bars in exterior joints are based on ductility requirements. Future research could suggest drift-based requirements for which both ductility and drift ratios could be accounted.

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APPENDIX I - DATABASE OF BEAM-COLUMN JOINTS CONSTRUCTED WITH HIGH-STRENGTH MATERIALS

Author	Specimen	Joint type	Column						Beam					Joint					
			f'_c	$b_c \times h_c$	Longitudinal reinforcement	Transverse reinforcement	ρ_g (%)	$P / f'_c A_g$	$b_b \times h_b$	Longitudinal reinforcement	Transverse reinforcement	d/h	ρ (%)	Transverse reinforcement	ρ_{jw} (%)	Joint core reinforce. ratio	Joint shear stress	Joint shear stress factor j	Failure type
Ehsani 1987[E2]	Unit 1	ext	64.7	340x340	4-D22(414Mpa) 4-D19(414Mpa)	3.14-D12@60(414Mpa)	2.32	0.02	300x480	4-D19(414Mpa) 6-D16(414Mpa)	2-D12@60(414Mpa)	1/17.89	0.86	3x4-D12@60(414Mpa)	1.14	0.275	5.36	0.67	B
	Unit 2	ext	67.3	340x340	4-D22(414Mpa) 4-D19(414Mpa)	3.14-D12@60(414Mpa)	2.32	0.04	300x480	6-D19(414Mpa) 4-D16(414Mpa)	2-D12@60(414Mpa)	1/17.90	1.3	3x4-D12@60(414Mpa)	1.14	0.275	6.54	0.80	B
	Unit 3	ext	64.7	300x300	4-D22(414Mpa) 4-D19(414Mpa)	3.14-D12@50(414Mpa)	2.98	0.07	259x439	6-D19(414Mpa) 4-D16(414Mpa)	2-D12@50(414Mpa)	1/15.79	1.3	3x4-D12@50(414Mpa)	1.46	0.34	7.61	0.95	BJ
	Unit 4	ext	67.3	300x300	4-D25(414Mpa) 4-D22(414Mpa)	3.14-D12@50(414Mpa)	3.99	0.05	259x439	6-D22(414Mpa) 4-D16(414Mpa)	2-D12@50(414Mpa)	1/13.64	1.62	3x4-D12@50(414Mpa)	1.46	0.34	9.45	1.15	BJ
Sugano 1990[S6]	J4-0	int	31.63	440x440	12-D22(416.32Mpa)	4-D9@100(960Mpa)	2.4	0.32	300x400	8-D22(401Mpa) 8-D22(401Mpa)	2-D6@75(888Mpa)	1/20.0	3.1	3-D9@50(960Mpa)	0.6	0.108	13.72	2.44	J
	J6-0	int	62.96	440x440	12-D22(416.32Mpa)	4-D9@100(960Mpa)	2.4	0.16	300x400	8-D22(401Mpa) 8-D22(401Mpa)	2-D6@75(888Mpa)	1/20.0	3.1	3-D9@50(960Mpa)	0.6	0.108	13.72	1.73	BJ
	J6-1	int	64.8	440x440	12-D22(416.32Mpa)	4-D9@100(960Mpa)	2.4	0.16	300x400	8-D22(401Mpa) 8-D22(401Mpa)	2-D6@75(888Mpa)	1/20.0	3.1	3-D9@50(960Mpa)	0.6	0.108	13.72	1.70	BJ
	J6-2	int	67.35	440x440	12-D22(416.32Mpa)	4-D9@100(960Mpa)	2.4	0.16	300x400	8-D22(416Mpa) 8-D22(416Mpa)	2-D6@75(888Mpa)	1/20.0	3.1	3-D9@50(960Mpa)	0.62	0.108	16.67	2.03	B

	J6-3	int	69.7	440x440	12-D22(416.32Mpa)	4-D9@100(960Mpa)	2.4	0.16	300x400	8-D22(416Mpa) 8-D22(416Mpa)	2-D6@75(888Mpa)	1/20.0	3.1	3-D9@50(960Mpa)	2.2	0.108	13.72	1.64	BJ
	J8-0	int	80.71	440x440	12-D22(416.32Mpa)	4-D9@100(960Mpa)	2.4	0.13	300x400	8-D22(401Mpa) 8-D22(401Mpa)	2-D6@75(888Mpa)	1/20.0	3.1	3-D9@50(960Mpa)	0.6	0.108	13.72	1.53	BJ
	J8H-0	int	83.36	440x440	12-D22(590.8Mpa)	4-D9@100(960Mpa)	2.4	0.13	300x400	8-D22(591Mpa) 8-D22(591Mpa)	2-D6@75(888Mpa)	1/20.0	3.1	3-D9@50(960Mpa)	0.6	0.108	19.62	2.15	BJ
	J8H-3	int	85.5	440x440	12-D22(590.8Mpa)	4-D9@100(960Mpa)	2.4	0.13	300x400	8-D22(591Mpa) 8-D22(591Mpa)	2-D6@75(888Mpa)	1/20.0	3.1	3-D9@50(960Mpa)	3.9	0.108	19.62	2.12	BJ
Alameddine 1991[A5]	LL8	ext	56.5	356x356	4-D25(427Mpa) 4-D22(427Mpa)	3-D13@88.9(446Mpa)	2.83	0.04	318x508	4-D25(427Mpa) 4-D25(427Mpa)	2-D13@88.9(446Mpa)	0.049	1.49	4x3-D13@88.9(446Mpa)	1.22	0.22	7.83	1.04	BJ
	LH8	ext	56.5	356x356	4-D25(427Mpa) 4-D22(427Mpa)	3-D13@88.9(446Mpa)	2.83	0.04	318x508	4-D25(427Mpa) 4-D25(427Mpa)	2-D13@88.9(446Mpa)	0.049	1.49	6x3-D13@88.9(446Mpa)	1.83	0.22	7.83	1.04	BJ
	HL8	ext	56.5	356x356	8-D25(427Mpa)	3-D13@88.9(446Mpa)	3.22	0.07	318x508	4-D29(427Mpa) 4-D29(427Mpa)	2-D13@88.9(446Mpa)	0.057	1.88	4x3-D13@88.9(446Mpa)	1.22	0.22	9.97	1.33	J
	HH8	ext	56.5	356x356	8-D25(427Mpa)	3-D13@88.9(446Mpa)	3.22	0.07	318x508	4-D29(427Mpa) 4-D29(427Mpa)	2-D13@88.9(446Mpa)	0.057	1.88	6x3-D13@88.9(446Mpa)	1.83	0.22	9.97	1.33	BJ
	LL11	ext	74.5	356x356	4-D25(427Mpa) 4-D22(427Mpa)	3-D13@88.9(446Mpa)	2.83	0.03	318x508	4-D25(427Mpa) 4-D25(427Mpa)	2-D13@88.9(446Mpa)	1/14.24	1.49	4x3-D13@88.9(446Mpa)	1.14	0.22	7.80	0.90	J
	LH11	ext	74.5	356x356	4-D25(427Mpa) 4-D22(427Mpa)	3-D13@88.9(446Mpa)	2.83	0.03	318x508	4-D25(427Mpa) 4-D25(427Mpa)	2-D13@88.9(446Mpa)	1/14.24	1.49	6x3-D13@88.9(446Mpa)	1.71	0.22	7.81	0.90	BJ
	HL11	ext	74.5	356x356	8-D25(427Mpa)	3-D13@88.9(446Mpa)	3.22	0.06	318x508	4-D29(427Mpa) 4-D29(427Mpa)	2-D13@88.9(446Mpa)	1/12.27	1.88	4x3-D13@88.9(446Mpa)	1.13	0.22	9.89	1.15	J
	HH11	ext	74.5	356x356	8-D25(427Mpa)	3-D13@88.9(446Mpa)	3.22	0.06	318x508	4-D29(427Mpa) 4-D29(427Mpa)	2-D13@88.9(446Mpa)	1/12.27	1.88	6x3-D13@88.9(446Mpa)	1.7	0.22	9.89	1.15	BJ

	LL14	ext	92.4	356x356	4-D25(427Mpa) 4-D22(427Mpa)	3-D13@88.9(446Mpa)	2.83	0.02	318x508	4-D25(427Mpa) 4-D25(427Mpa)	2-D13@88.9(446Mpa)	1/14.24	1.49	4x3-D13@88.9(446Mpa)	1.22	0.22	7.80	0.81	BJ
	LH14	ext	92.4	356x356	4-D25(427Mpa) 4-D22(427Mpa)	3-D13@88.9(446Mpa)	2.83	0.02	318x508	4-D25(427Mpa) 4-D25(427Mpa)	2-D13@88.9(446Mpa)	1/14.24	1.49	6x3-D13@88.9(446Mpa)	1.83	0.22	7.80	0.81	BJ
	HH14	ext	92.4	356x356	8-D25(427Mpa)	3-D13@88.9(446Mpa)	3.22	0.04	318x508	4-D29(427Mpa) 4-D29(427Mpa)	2-D13@88.9(446Mpa)	1/12.27	1.88	6x3-D13@88.9(446Mpa)	1.83	0.22	9.92	1.03	BJ
Fujii 1991[F2]	A1	int	40.2	220x220	16-D13(644Mpa)	2-φ6@50(291Mpa)	4.19	0.08	160x250	8-D10(1069Mpa)	2-φ6@50(291Mpa)	1/22.0	1.68	3x2-φ6@50(291Mpa)	0.41	0.102	9.86	1.55	J
	A2	int	40.2	220x220	16-D13(387Mpa)	2-φ6@50(291Mpa)	4.19	0.08	160x250	8-D10(409Mpa)	2-φ6@50(291Mpa)	1/22.0	1.68	3x2-φ6@50(291Mpa)	0.41	0.102	9.08	1.43	J
	A3	int	40.2	220x220	16-D13(644Mpa)	2-φ6@50(291Mpa)	4.19	0.23	160x250	8-D10(1069Mpa)	2-φ6@50(291Mpa)	1/22.0	1.68	3x2-φ6@50(291Mpa)	0.41	0.102	9.86	1.55	J
	A4	int	40.2	220x220	16-D13(644Mpa)	2-φ6@50(291Mpa)	4.19	0.23	160x250	8-D10(1069Mpa)	2-φ6@50(291Mpa)	1/22.0	1.68	4x4-φ6@50(291Mpa)	1.1	0.102	9.86	1.55	J
	B1	ext	30	220x220	12-D13(387Mpa)	2-φ6@50(291Mpa)	3.14	0.07	160x250	8-D10(1069Mpa)	2-φ6@50(291Mpa)	1/22.0	1.68	3x2-φ6@50(291Mpa)	0.41	0.102	5.89	1.08	J
	B2	ext	30	220x220	12-D13(387Mpa)	2-φ6@50(291Mpa)	3.14	0.07	160x250	8-D10(409Mpa)	2-φ6@50(291Mpa)	1/22.0	1.68	3x2-φ6@50(291Mpa)	0.41	0.102	5.12	0.93	BJ
	B3	ext	30	220x220	12-D13(387Mpa)	2-φ6@50(291Mpa)	3.14	0.24	160x250	8-D10(1069Mpa)	2-φ6@50(291Mpa)	1/22.0	1.68	3x2-φ6@50(291Mpa)	0.41	0.102	6.53	1.19	J
	B4	ext	30	220x220	12-D13(387Mpa)	2-φ6@50(291Mpa)	3.14	0.24	160x250	8-D10(1069Mpa)	2-φ6@50(291Mpa)	1/22.0	1.68	4x4-φ6@50(291Mpa)	1.1	0.102	6.87	1.25	J
Kitayama 1992[K1]	I1	int	98.8	300x300	16-D19(746.5Mpa)	2-U6.4@40(1308.7Mpa)	5.1	0.02	200x300	8-D16(798.5Mpa) 8-D16(798.5Mpa)	2-U6.4@35(1308.7Mpa)	1/18.75	3.34	3x3-R6(360Mpa)	0.41	0.093	19.88	2.00	J
	I2	int	98.8	300x300	16-D19(746.5Mpa)	2-U6.4@40(1308.7Mpa)	5.1	0.02	200x300	8-D16(798.5Mpa) 8-D16(798.5Mpa)	2-U6.4@35(1308.7Mpa)	1/18.75	3.65	2x4-R6(360Mpa)	0.39	0.126			J
	I3	int	41.4	300x300	16-D16(361Mpa)	4-D6@50(358.1Mpa)	3.54	0.02	200x300	6-D16(361Mpa) 4-D16(361Mpa)	2-D6@40(358.1Mpa)	1/18.75	1.53	3x3-R6(360Mpa)	0.37	0.093	8.11	1.26	BJ
	I5	int	85.4	300x300	16-D16(533.7Mpa)	4-D6@50(395.3Mpa)	3.54	0.02	200x300	6-D13(769.1Mpa) 3-D13(769.1Mpa)	2-D6@40(395.3Mpa)	1/23.08	0.73	3x4-R5.5(250.2Mpa)	0.42	0.107	9.43	1.02	BJ
	I6	int	85.4	300x300	16-D16(533.7Mpa)	4-D6@50(395.3Mpa)	3.54	0.02	200x300	3-D19(772Mpa) 2-D19(772Mpa)	2-D6@40(395.3Mpa)	1/15.79	1.1	3x4-R5.5(250.2Mpa)	0.42	0.107	11.18	1.21	BJ
Oka	J-1	int	81.2	300x300	24-D13(638Mpa)	2-D6@50(400Mpa)	3.38	0.11	240x300	9-D13(638Mpa) 7-D13(638Mpa)	2-D6@50(400)	1/23.08	1.84	5x2-D6@50(400Mpa)	0.5	0.063	14.20	1.58	BJ

1992[01]	J-2	int	81.2	300x300	24-U13(1456Mpa)	2-D6@50(400Mpa)	3.38	0.11	240x300	8-U13(1456Mpa) 8-U13(1456Mpa)	2-D6@50(400)	1/23.08	1.84	5x2-D6@50(400Mpa)	0.5	0.063	15.30	1.70	J
	J-3	int	81.2	300x300	24-U13(1456Mpa)	2-D6@50(400Mpa)	3.38	0.11	240x300	8-U13(1456Mpa) 8-U13(1456Mpa)	2-D6@50(400)	1/23.08	1.84	5x2-D6@50(400Mpa)	0.5	0.063	18.10	2.01	J
	J-4	int	72.8	300x300	24-D13(515Mpa)	2-D6@50(400Mpa)	3.38	0.11	240x300	10-D13(515Mpa) 10-D13(515Mpa)	2-D6@50(400)	1/23.08	1.84	5x2-D6@50(400Mpa)	0.5	0.063	14.50	1.70	BJ
	J-5	int	72.8	300x300	24-D13(515Mpa)	2-D6@50(400Mpa)	3.38	0.11	240x300	9-D13(515Mpa) 7-D13(515Mpa)	2-D6@50(400)	1/23.08	1.84	5x2-D6@50(400Mpa)	0.5	0.063	16.30	1.91	J
	J-6	int	79.2	300x300	24-D13(676Mpa)	2-D6@50(400Mpa)	3.38	0.12	240x300	9-D13(676Mpa) 7-D13(676Mpa)	2-D6@50(400)	1/23.08	1.84	5x2-D6@50(400Mpa)	0.5	0.063	15.10	1.70	BJ
	J-7	int	79.2	300x300	24-D13(676Mpa)	2-D6@50(400Mpa)	3.38	0.12	240x300		2-D6@50(400)	1/23.08	1.37	5x2-D6@50(400Mpa)	0.46	0.063	12.00	1.35	B
	J-8	int	79.2	300x300	24-D19(370Mpa)	2-D6@50(400Mpa)	3.38	0.12	240x300	9-D19(370Mpa) 7-D19(370Mpa)	2-D6@50(400)	1/15.79	1.84	5x2-D6@50(400Mpa)	0.5	0.063	17.10	1.92	J
	J-9	int	79.2	300x300	24-D13(676Mpa)	2-D6@50(400Mpa)	3.38	0.12	240x300	7-D13(676Mpa) 7-D13(676Mpa)	2-D6@50(400)	1/23.08	1.84	5x2-D6@50(400Mpa)	0.5	0.063	16.10	1.81	B
	J-10	int	39.2	300x300	24-D13(700Mpa)	2-D6@50(400Mpa)	3.38	0.12	240x300	9-D13(700Mpa) 7-D13(700Mpa)	2-D6@50(400)	1/23.08	1.84	5x2-D6@50(400Mpa)	0.5	0.063	10.80	1.72	J
	J-11	int	39.2	300x300	24-D19(372Mpa)	2-D6@50(400Mpa)	3.38	0.12	240x300	9-D19(372Mpa) 7-D19(372Mpa)	2-D6@50(400)	1/15.79	1.84	5x2-D6@50(400Mpa)	0.5	0.063	12.70	2.03	J
Noguchi 1992 [N2]	OKJ-1	int	70	300x300	20-D13(718)	2-D6@40(955Mpa)	2.82	0.12	200x300	9-D13(718Mpa) 7-D13(718Mpa)	2-D6@50(955Mpa)	1/23.08	2.21	3x2-D6@50(955Mpa)	0.26	0.063	12.63	1.51	BJ
	OKJ-3	int	107	300x300	22-D13(718)	2-D6@40(955Mpa)	3.1	0.12	200x300	10-D13(718Mpa) 10-D13(718Mpa)	2-D6@50(955Mpa)	1/23.08	2.53	3x4-D6@50(955Mpa)	0.53	0.12	15.72	1.52	J
	OKJ-4	int	70	300x300	20-D13(718)	2-D6@40(955Mpa)	2.82	0.12	200x300	9-D13(718Mpa) 7-D13(718Mpa)	2-D6@50(955Mpa)	1/23.08	2.21	3x4-D6@50(955Mpa)	0.53	0.12	12.72	1.52	BJ
	OKJ-5	int	70	300x300	24-D13(718)	2-D6@40(955Mpa)	3.83	0.12	200x300	10-D13(718Mpa) 10-D13(718Mpa)	2-D6@50(955Mpa)	1/23.08	2.53	3x2-D6@50(955Mpa)	0.26	0.063	13.39	1.60	J
	OKJ-6	int	53.5	300x300	20-D13(718)	2-D6@40(955Mpa)	2.82	0.12	200x300	8-D13(718Mpa) 7-D13(718Mpa)	2-D6@50(955Mpa)	1/23.08	1.96	3x2-D6@50(955Mpa)	0.26	0.063	12.00	1.64	J
	MKJ-1	int	86	300x300	12-D10(644Mpa)	2-D6@80(675Mpa)	0.95	0.1	200x300	2-D19(771Mpa) 2-D19(771Mpa)	2-D6@90(675Mpa)	1/15.79	1.17	3x4-D6@50(675Mpa)	0.53	0.125	11.78	1.27	BJ
	MKJ-2	int	86	300x300	12-D16(718Mpa)	2-D6@50(675Mpa)	2.65	0.1	200x300	3-D19(771Mpa) 3-D19(771Mpa)	2-D6@60(675Mpa)	1/15.79	1.76	3x4-D6@50(675Mpa)	0.53	0.125	23.18	2.50	BJ

	MKJ-3	int	100.4	300x300	12-D13(794Mpa)	2-D6@65(675Mpa)	1.69	0.1	200x300	2-D22(742Mpa) 2-D19(771Mpa)	2-D6@50(675Mpa)	1/13.64	1.55	3x4-D6@50(675Mpa)	0.53	0.125	20.64	2.06	BJ
	MKJ-4	int	100.4	300x300	12-D19(771Mpa)	2-D6@40(675Mpa)	3.82	0.1	200x300	3-D22(742Mpa) 2-D22(742Mpa)	2-D6@50(675Mpa)	1/13.64	2.34	3x4-D6@50(675Mpa)	0.53	0.125	20.04	2.00	BJ
Teraoka 1994[T3]	HNO.1	int	94	400x400	8-HD22(603Mpa) + 4-HD19(635Mpa)	4-D10@50(343Mpa)	2.65	0.2	300x400	8-HD16(611Mpa) 8-HD16(611Mpa)	4-HD6@80(827Mpa)	1/25.0	1.63	6x4-HD8@50(681Mpa)	1	0.125	11.93	1.23	BJ
	HNO.3	int	94	400x400	8-D22(441Mpa) + 4-D19(442Mpa)	4-D10@50(343Mpa)	2.65	0.2	300x400	8-D22(441Mpa) 8-D22(441Mpa)	4-HD6@60(827Mpa)	1/18.18	3.18	6x4-HD8@50(681Mpa)	1	0.125	15.80	1.63	BJ
	HNO.4	int	94	400x400	8-HD22(603Mpa) + 4-HD19(635Mpa)	4-D10@50(343Mpa)	2.65	0.2	300x400	8-HD22(603Mpa) 8-HD22(603Mpa)	4-HD6@50(827Mpa)	1/18.18	3.18	6x4-HD8@50(681Mpa)	1	0.125	18.32	1.89	BJ
	HNO.5	int	108	400x400	8-HD22(603Mpa) + 4-HD19(635Mpa)	4-D10@50(343Mpa)	2.65	0.2	300x400	8-HD19(635Mpa) 8-HD16(611Mpa)	4-HD6@60(827Mpa)	1/21.05	1.08	6x4-HD8@50(681Mpa)	1	0.125	12.78	1.23	BJ
	HNO.6	int	108	400x400	8-HD22(603Mpa) + 4-HD19(635Mpa)	4-D10@50(343Mpa)	2.65	0.2	300x400	8-HD22(603Mpa) 8-HD22(603Mpa)	4-HD6@50(827Mpa)	1/18.18	3.18	6x4-HD8@50(681Mpa)	1	0.125	18.60	1.79	BJ
Teraoka 2004[T2]	HJ-1	int	54.3	400x400	12-D19(382Mpa)	4-D10@100(347Mpa)	1.49	0.18	300x400	4-D19(382Mpa) 4-D19(382Mpa)	4-D6@100(312Mpa)	1/21.05	1.11	7x2-D10@50(347Mpa)	0.73	0.098	5.67	0.77	B
	HJ-2	int	54.3	400x400	12-D19(382Mpa)	4-D10@100(347Mpa)	1.49	0.18	300x400	4-HD16(624Mpa) 4-HD16(624Mpa)	4-D6@100(312Mpa)	1/25.0	0.77	7x2-D10@50(347Mpa)	0.73	0.098	6.34	0.86	B
	HJ-3	int	54.3	400x400	12-D19(382Mpa)	4-D10@100(347Mpa)	1.49	0.18	300x400	2-UD19(858Mpa) 2-UD19(858Mpa)	2-D10@110(347Mpa)	1/21.05	0.56	7x2-D10@50(347Mpa)	0.73	0.098	5.31	0.72	BJ
	HJ-4	int	54.3	400x400	12-HD19(645Mpa)	4-D10@100(347Mpa)	1.49	0.18	300x400	6-D19(382Mpa) 6-D19(382Mpa)	4-HD6@120(762Mpa)	1/21.05	1.72	7x2-D10@50(347Mpa)	0.73	0.098	7.88	1.07	BJ
	HJ-5	int	54.3	400x400	12-HD19(645Mpa)	4-D10@100(347Mpa)	1.49	0.18	300x400	4-HD19(645Mpa) 4-HD19(645Mpa)	4-HD6@120(762Mpa)	1/21.05	1.11	7x2-D10@50(347Mpa)	0.73	0.098	8.47	1.15	BJ
	HJ-6	int	54.3	400x400	12-HD19(645Mpa)	4-D10@100(347Mpa)	1.49	0.18	300x400	3-UD19(858Mpa) 3-UD19(858Mpa)	3-HD6@90(762Mpa)	1/21.05	0.83	7x2-D10@50(347Mpa)	0.73	0.098	8.25	1.12	BJ
	HJ-7	int	83.4	400x400	12-HD19(645Mpa)	4-D10@100(347Mpa)	1.49	0.2	300x400	6-D22(422Mpa) 6-D22(422Mpa)	4-HD6@90(762Mpa)	1/18.18	2.33	7x4-HD8@50(681Mpa)	1.05	0.125	10.96	1.20	BJ
	HJ-8	int	83.4	400x400	12-HD19(645Mpa)	4-D10@100(347Mpa)	1.49	0.2	300x400	4-HD22(599Mpa) 4-HD22(599Mpa)	4-HD6@90(762Mpa)	1/18.18	1.5	7x4-HD8@50(681Mpa)	1.02	0.125	11.14	1.22	BJ
	HJ-9	int	83.4	400x400	12-HD19(645Mpa)	4-D10@100(347Mpa)	1.49	0.2	300x400	4-UD19(858Mpa) 4-UD19(858Mpa)	4-HD6@90(762Mpa)	1/21.05	1.11	7x4-HD8@50(681Mpa)	1.02	0.125	13.79	1.51	BJ
	HJ-10	int	93.8	400x400	8-HD22(604Mpa) + 4-HD19(634Mpa)	4-D10@50(343Mpa)	2.43	0.167	300x400	8-HD16(611Mpa) 8-HD16(611Mpa)	4-HD6@90(762Mpa)	1/25.0	1.62	7x4-HD8@50(681Mpa)	1.07	0.125	12.30	1.27	BJ

	HJ-11	int	93.8	400x400	8-D22(441Mpa) + 4-D19(442Mpa)	4-D10@50(343Mpa)	2.43	0.167	300x400	8-D22(441Mpa) 8-D22(441Mpa)	4-HD6@90(762Mpa)	1/18.18	3.16	7x4-HD8@50(681Mpa)	1.07	0.125	16.08	1.66	BJ
	HJ-12	int	93.8	400x400	8-HD22(604Mpa) + 4-HD19(634Mpa)	4-D10@50(343Mpa)	2.43	0.167	300x400	8-HD22(604Mpa) 8-HD22(604Mpa)	4-HD6@90(762Mpa)	1/18.18	3.16	7x4-HD8@50(681Mpa)	1.07	0.125	18.79	1.94	BJ
	HJ-13	int	107.6	400x400	8-HD22(604Mpa) + 4-HD19(634Mpa)	4-D10@100(343Mpa)	2.43	0.125	300x400	8-HD19(634Mpa) +4-HD16(611Mpa) 8-HD19(634Mpa) +4-HD16(611Mpa)	4-HD6@90(762Mpa)	1/21.05	1.88	7x4-HD8@50(681Mpa)	1.07	0.125	13.17	1.27	BJ
	HJ-14	int	107.6	400x400	8-HD22(604Mpa) + 4-HD19(634Mpa)	4-D10@100(343Mpa)	2.43	0.125	300x400	8-HD22(604Mpa) 8-HD22(604Mpa)	4-HD6@90(762Mpa)	1/18.18	3.16	7x4-HD8@50(681Mpa)	1.07	0.125	18.36	1.77	BJ
	HJ-15	int	146.3	450x450	12-UD22(776Mpa)	4-UD8@40(986Mpa)	2.29	0.2	320x450	4-UD22(776Mpa) 4-UD22(776Mpa)	4-HD8@70(986Mpa)	1/20.45	1.41	7x2-UD8@40(986Mpa)	0.45	0.0496	14.03	1.16	BJ
	HJ-16	int	146.3	450x450	12-UD22(776Mpa)	4-UD8@40(986Mpa)	2.29	0.2	320x450	6-UD22(776Mpa) 6-UD22(776Mpa)	4-HD8@50(986Mpa)	1/20.45	2.18	7x2-UD8@40(986Mpa)	0.47	0.0496	20.20	1.67	BJ
Hwang 2005[H7]	3T44	ext	76.8	420x420	8-D32(659Mpa)	3-D13@97(498Mpa)	3.71	0.014	320x450	4-D25(430Mpa) 4-D25(430Mpa)	4-D13@97(436Mpa)	1/16.8	1.66	3x3x2-D13@107(498Mpa)	1.73	0.21	7.92	0.90	B
	3T3	ext	69	420x420	8-D32(659Mpa)	3-D13@97(498Mpa)	3.71	0.016	320x450	4-D25(430Mpa) 4-D25(430Mpa)	4-D13@97(436Mpa)	1/16.8	1.66	3x3-D10@107(471Mpa)	0.48	0.125	8.42	1.01	BJ
	2T4	ext	71	420x420	8-D32(659Mpa)	2-D13@97(498Mpa)	3.71	0.016	320x450	4-D25(430Mpa) 4-D25(430Mpa)	4-D13@97(436Mpa)	1/16.8	1.66	2x2-D13@160(498Mpa)	0.38	0.14	8.04	0.95	BJ
	1T44	ext	72.8	420x420	8-D32(659Mpa)	2-D13@97(498Mpa)	3.71	0.015	320x450	4-D25(430Mpa) 4-D25(430Mpa)	4-D13@97(436Mpa)	1/16.8	1.66	1x2-D13@160(498Mpa)	0.19	0.14	7.73	0.91	BJ
	3T4	ext	75.2	450x450	8-D32(690Mpa)	3-D13@97(498Mpa)	3.24	0.013	320x450	4-D25(491Mpa) 4-D25(491Mpa)	4-D13@97(436Mpa)	1/18.0	1.66	3x3-D13@107(498Mpa)	0.81	0.196	7.71	0.89	B
	2T5	ext	76.6	450x450	8-D32(690Mpa)	2-D13@97(498Mpa)	3.24	0.013	320x450	4-D25(491Mpa) 4-D25(491Mpa)	4-D13@97(436Mpa)	1/18.0	1.66	2x2-D16@160(446Mpa)	0.55	0.198	8.07	0.92	B
	1T55	ext	69.7	450x450	8-D32(690Mpa)	2-D13@97(498Mpa)	3.24	0.014	320x450	4-D25(491Mpa) 4-D25(491Mpa)	4-D13@97(436Mpa)	1/18.0	1.66	1x2-D16@160(446Mpa)	0.28	0.198	7.82	0.94	B
Li 2014[L2]	NS1	int	60	450x300	4T28 + 2T24 + 2T20 (500Mpa)	4-D10@60(360Mpa)		0	250x500	7-D12(512Mpa) 7-D12(512Mpa)	2-D6@90(360Mpa)	1/37.5	0.68	5x4-D10@65(360Mpa)		0.209	4.78	0.62	BJ
	NS2	int	60	450x300	4T24 + 4T16 (500Mpa)	4-D10@60(360Mpa)		0	250x500	4-D16(512Mpa) 2-D16(512Mpa)	2-D6@90(360Mpa)	1/28.1	0.69	5x4-D10@65(360Mpa)		0.142	4.65	0.60	BJ
	NS3	int	60	450x300	4T32 + 4T20 (500Mpa)	4-D10@90(360Mpa)		0	250x500	4-D16(512Mpa) 4-D16(512Mpa)	2-D6@90(360Mpa)	1/28.1	0.69	5x4-D10@65(360Mpa)		0.209	3.62	0.47	BJ

NS4	int	60	450x300	4T28 + 4T16 (500Mpa)	4-D10@90(360Mpa)		0	250x500	2-D20(512Mpa) 2-D16(512Mpa)	2-D6@90(360Mpa)	1/22.5	0.54	4x4-D10@85(360Mpa)		0.209	3.80	0.49	BJ
As1	int	60	450x300	4T28 + 2T24 + 2T20 (500Mpa)	4-D10@60(360Mpa)		0.3	250x500	7-D12(512Mpa) 7-D12(512Mpa)	2-D6@90(360Mpa)	1/37.5	0.68	5x4-D10@65(360Mpa)		0.209	4.97	0.64	BJ
As2	int	60	450x300	4T24 + 4T16 (500Mpa)	4-D10@60(360Mpa)		0.3	250x500	4-D16(512Mpa) 2-D16(512Mpa)	2-D6@90(360Mpa)	1/28.1	0.69	5x4-D10@65(360Mpa)		0.142	5.04	0.65	BJ
As3	int	60	450x300	4T32 + 4T20 (500Mpa)	4-D10@90(360Mpa)		0.3	250x500	4-D16(512Mpa) 4-D16(512Mpa)	2-D6@90(360Mpa)	1/28.1	0.69	5x4-D10@65(360Mpa)		0.209	3.14	0.41	BJ
As4	int	60	450x300	4T28 + 4T16 (500Mpa)	4-D10@90(360Mpa)		0.3	250x500	2-D20(512Mpa) 2-D16(512Mpa)	2-D6@90(360Mpa)	1/22.5	0.54	4x4-D10@85(360Mpa)		0.209	3.14	0.41	BJ

APPENDIX II – STRAIN OF STEEL REINFORCEMENT AT THE JOINT REGION

Figure II.1 Strain of reinforcement of Specimen IN80

Figure II.2 Strain of reinforcement of Specimen IH80

Figure II.3 Strain of reinforcement of Specimen IH80A

Figure II.4 Strain of reinforcement of Specimen IN100

Figure II.5 Strain of reinforcement of Specimen IH100

Figure II.6 Strain of reinforcement of Specimen IH60

Figure II.7 Strain of reinforcement of Specimen IH60A

Figure II.8 Strain of reinforcement of Specimen EN80

Figure II.9 Strain of reinforcement of Specimen EH80

Figure II.10 Strain of reinforcement of Specimen EH80A

Figure II.11 Strain of reinforcement of Specimen EH60

Figure II.12 Strain of reinforcement of Specimen EH60A

APPENDIX III – DIANA INPUT AND ANALYSIS FILES

DIANA Input Data File (*.DAT)

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FEMGEN MODEL      : IN80
ANALYSIS TYPE     : Structural 3D
'UNITS'
LENGTH  MM
TIME    SEC
TEMPER  KELVIN
FORCE   N
'COORDINATES'
  1  0.000000E+00  1.260000E+03  0.000000E+00
  2  1.500000E+02  1.260000E+03  0.000000E+00
  3  3.000000E+02  1.260000E+03  0.000000E+00
  4  4.500000E+02  1.260000E+03  0.000000E+00
      .
      .
      .
3663  1.500000E+02  8.400000E+02  1.250000E+01
3664  1.500000E+02  9.800000E+02  1.250000E+01
3665  1.500000E+02  1.120000E+03  1.250000E+01
'ELEMENTS'
CONNECTIVITY
  1 CHX60 39 55 42 117 89 109 92 116 52 59 106 113 1 9 2 13 6 16 5 12
  2 CHX60 42 62 45 118 86 102 89 117 59 66 99 106 2 10 3 14 7 17 6 13
  3 CHX60 45 69 48 79 76 95 86 118 66 73 83 99 3 11 4 15 8 18 7 14
  4 CHX60 38 54 41 119 88 108 91 115 51 58 105 112 39 55 42 117 89 109 92 116
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      .
      .
583 CHX60 3573 3590 3574 3582 455 505 454 3581 3643 3642 497 496 3601
      3634 3600 3665 447 488 446 3664
584 CHX60 3574 3591 1 12 5 506 455 3582 3642 9 16 497 3600 3633 2
      13 6 489 447 3665

MATERIALS
/ 1-490 531-584 / 1
/ 491-530 / 2
'REINFORCEMENTS'
LOCATI
2 BAR
  LINE  4.700000E+01  0.000000E+00  4.700000E+01
        4.700000E+01  3.300000E+03  4.700000E+01
4 BAR
  LINE  2.250000E+02  0.000000E+00  4.700000E+01
        2.250000E+02  3.300000E+03  4.700000E+01
6 BAR
  LINE  4.030000E+02  0.000000E+00  4.700000E+01
        4.030000E+02  3.300000E+03  4.700000E+01

```

•
•
•

MATERIALS

/ 2 6 10 14 / 3	:Column longitudinal corner bars:
/ 4 8 12 16 / 4	:Column longitudinal middle bars:
/ 45-788 / 5	:Column hoops:
/ 797-868 / 6	:Joint hoops:
/ 18 20 22 24 26 28 / 7	:Beam longitudinal bars:
/ 873-1252 / 8	:Beam stirrups:

GEOMETRY

/ 2 6 10 14 / 3	:Column longitudinal corner bars:
/ 4 8 12 16 / 4	:Column longitudinal middle bars:
/ 45-788 / 5	:Column hoops:
/ 797-868 / 6	:Joint hoops:
/ 18 20 22 24 26 28 / 7	:Beam longitudinal bars:
/ 873-1252 / 8	:Beam stirrups:

'MATERIALS'

```

1 YOUNG  2.000000E+04
  POISON  1.600000E-01
  TOTCRK ROTATE
  TENCRCV LINEAR
  TENSTR  1.000000E+01
  GF1     2.000000E-01
  COMCRV PARABO
  COMSTR  8.000000E+01
  GC      3.000000E+01
  CNFCRV VECCHI
  REDCRV VC1993
  SHRCRV CONSTA
  BETA    1.000000E-02
2 YOUNG  1.000000E+05
  POISON  2.000000E-01
3 YOUNG  2.000000E+05
  POISON  2.000000E-01
  YIELD MENEGP
  YLDVAL  550 0.008 19.5 18.5 0.1 0.0 0.0
4 YOUNG  2.000000E+05
  POISON  2.000000E-01
  YIELD MENEGP
  YLDVAL  550 0.008 19.5 18.5 0.1 0.0 0.0
5 YOUNG  2.000000E+05
  POISON  2.000000E-01
  YIELD MENEGP
  YLDVAL  550 0.008 19.5 18.5 0.1 0.0 0.0
6 YOUNG  2.000000E+05
  POISON  2.000000E-01

```



```

YIELD MENEGP
YLDVAL 550 0.008 19.5 18.5 0.1 0.0 0.0
7 YOUNG 2.000000E+05
POISON 2.000000E-01
YIELD MENEGP
YLDVAL 550 0.008 19.5 18.5 0.1 0.0 0.0
8 YOUNG 2.000000E+05
POISON 2.000000E-01
YIELD MENEGP
YLDVAL 550 0.008 19.5 18.5 0.1 0.0 0.0
'GEOMETRY'
3 CROSSE 4.908738E+02
4 CROSSE 2.010619E+02
5 CROSSE 7.853900E+01
6 CROSSE 7.853900E+01
7 CROSSE 2.010619E+02
8 CROSSE 7.853900E+01

'GROUPS'
REINFO
1 RS1 / 61-68 /
2 RS2 / 77-84 /
3 RS3 / 93-100 /
4 RS4 / 109-116 /
.
.
.

99 RS97 / 1241-1244 /
100 RS98 / 1249-1252 /

'SUPPORTS'
/3170 / TR 1 TR 2 TR 3
/3283 / TR 1 TR 2 TR 3
/3380 3378 3366 3368 3326 3328 3316 3314 / TR 2 TR 3

'LOADS'
CASE 1
DEFORM
3283 TR 1 0.100000E+01
'DIRECTIONS'
1 1.000000E+00 0.000000E+00 0.000000E+00
2 0.000000E+00 1.000000E+00 0.000000E+00
3 0.000000E+00 0.000000E+00 1.000000E+00
'END'

```

DIANA Analysis Command File (*.DCF)

```

*FILOS
INITIA
*NONLIN
BEGIN EXECUTE
BEGIN LOAD
LOADNR 1
STEPS EXPLIC SIZES 5.46(3) -5.46(6) 5.46(3) 8.68(3) -8.68(6) 8.68(3)\
                    8.625(4) -8.625(8) 8.625(4) 9.6(5) -9.6(10) 9.6(5) \
                    9.6(5) -9.6(10) 9.6(5) 12.8(5) -12.8(10) 12.8(5) \
                    12.8(5) -12.8(10) 12.8(5) 13.5(6) -13.5(12) 13.5(6) \
                    13.5(6) -13.5(12) 13.5(6) 16.67(6) -16.67(12) 16.67(6) \
                    16.67(6) -16.67(12) 16.67(6) 19.25(6) -19.25(12) 19.25(6) \
                    19.25(6) -19.25(12) 19.25(6) 16.5(8) -16.5(16) 16.5(8) \
                    16.5(8) -16.5(16) 16.5(8)

END LOAD
BEGIN ITERAT
BEGIN CONVER
DISPLA OFF
BEGIN ENERGY
CONTIN
TOLCON 0.001
END ENERGY
FORCE OFF
END CONVER
MAXITE 300
METHOD SECANT
END ITERAT
END EXECUT
BEGIN OUTPUT
FILE "Output1"
DISPLA TOTAL TRANSL GLOBAL
FORCE REACTI TRANSL GLOBAL
STRAIN CRACK GREEN
STRAIN PLASTI GREEN GLOBAL
STRAIN ELASTI GREEN GLOBAL
STRAIN TOTAL GREEN GLOBAL
STRESS TOTAL CAUCHY GLOBAL
END OUTPUT
BEGIN OUTPUT
TABULA
FILE "Output2"
SELECT NODES 3283 /
DISPLA TOTAL TRANSL GLOBAL
FORCE REACTI TRANSL GLOBAL
END OUTPUT
BEGIN TYPE
BEGIN PHYSIC
CORROS OFF
MATURI OFF
SHRINK OFF
TEMPER OFF

```

END PHYSIC
END TYPE
*END

APPENDIX IV – STRUT AND TIE MODELING PROCEDURE FOR BEAM-COLUMN JOINTS

The procedure to find the internal force flow in an interior beam-column joint is explained in this appendix. The strut-and-tie modeling of Specimen IH80 will be studied. The design parameters are given as $f'_c = 80\text{MPa}$, $P = 0$, and $V_{sh}/V_{jh} = 0.54$. The beams are reinforced with 4T16 top and 2T16 bottom longitudinal bars with $f_y = 700\text{MPa}$. Five struts are used to model the internal force flow in the joint. Struts are designated as a-a', b-b', c-c', d-d' and e-e'. The steps of the modeling calculation are presented as follows:

Figure IV.1 An Example of STM Analysis of Joints

Step 1: Calculate the tensile forces in the beam top and bottom bars at over-strength.

$$T_{b,1} = 1.25 \times 0.700 \text{KN} / \text{mm}^2 \times 402.12 \text{mm}^2 = 351.8 \text{KN}$$

$$T_{b,2} = 1.25 \times 0.700 \text{KN} / \text{mm}^2 \times 2 \times 402.12 \text{mm}^2 = 703.7 \text{KN}$$

Step 2: Calculate the horizontal joint shear force, V_{jh}

The bending moment at over-strength in the adjacent beams can be approximated as

$$M^+ = 351.8KN \times 0.414m = 145.6KN.m$$

$$M^- = 703.7KN \times 0.414m = 291.3KN.m$$

The corresponding beam shear forces are calculated assuming that the critical section coincide with the outmost layer of the longitudinal column reinforcement.

$$V_b^+ = 145.6KN.m / (2.375m + 0.048m) = 60.1KN$$

$$V_b^- = 291.3KN.m / (2.375m + 0.048m) = 120.2KN$$

The column shear force required for equilibrium is

$$V_{col} = 90.1KN \times 5.2m / 3.3m = 142.0KN$$

Hence, the horizontal joint shear force is

$$V_{jh} = 351.8 + 703.7 - 142.0 = 913.5KN$$

Step 3: Calculate the shear force V_{sh} carried by the horizontal joint reinforcement.

$$V_{sh} = 0.54 \times 913.5KN = 498KN$$

Assume the confining force of the joint hoops is uniformly distributed over 85% of the distance between beam top and bottom longitudinal bars. Thos distance is equal to $\pm 176mm$ from the center of the joint. The magnitude of the pressure is $498KN / (2 \times 176mm) = 1.41KN / mm$.

Step 4: Determine the potential bond forces at nodes “a” and “b” at the intersection of the longitudinal beam and column bars following the bond stress law proposed in

Chapter 9. The bond forces are calculated as follows:

$$\tau_u = 3.67KN \text{ Then:}$$

$$\text{Top bars) } U_a = 3.67KN \times 356 \times (3.14 \times 16 \times 4) / 1000 = 262.7KN$$

$$\text{Bottom bars) } U_a = 3.67KN \times 356 \times (3.14 \times 16 \times 2) / 1000 = 131.3KN$$

Step 5: Find the column reinforcing steel and concrete forces due to combined bending moment an axial load. Calculate the column bending moment at the level of the longitudinal beam reinforcement:

$$M_{col} = 142.0KN \times (1.4m + 0.04m) = 205.0KN.m$$

The forces in longitudinal column bars and in concrete due to M_{col} can be obtained from a section moment-curvature analysis. The force values and their location are shown in **Figure IV.1**.

Step 6: Determine the geometry and forces in struts a-a' and e-e'.

The horizontal component of the diagonal strut a-a' is

$$H_{aa'} = U_a = 262.7 \text{ KN}$$

Force $H_{aa'}$ has to be balanced by the uniform pressure from the joint hoops. Thus, a fraction of the uniform pressure with a resultant equal to $H_{aa'}$ is taken by this strut.

The position of the resulting force measured from joint mid-depth is

$$y_{a'} = 176 - (262.7 \text{ KN}) / 2 / 1.41 \text{ KN/mm} = 82.8 \text{ mm}$$

Once the geometry of strut a-a' is determined, the vertical force carried by the strut is obtained from the geometry.

$$V_{aa'} = (207 - 82.8) \text{ mm} \times 262.7 \text{ KN} / (178 \text{ mm}) = 183.3 \text{ KN}$$

The force in the strut is

$$S_{aa'} = \sqrt{(262.7 \text{ KN})^2 + (183.3 \text{ KN})^2} = 320.3 \text{ KN (compression)}$$

The tensile force in the column interior bar at node "a" due to bending in the column and strut a-a' is

$$T_{c,a} = 34 + 183.3 = 217.3 \text{ KN}$$

This tensile force cannot exceed the yield force of the column interior bars that is

$$= 2 \times (\pi \times 20^2 / 4) \text{ mm}^2 \times 550 \text{ MPa} = 345 \text{ KN}$$

$$T_{c,a} = 217.3 \text{ KN} < 345 \text{ KN}, \text{ satisfactory.}$$

A similar procedure is conducted for strut e-e':

$$H_{ee'} = U_a = 131.3 \text{ KN}$$

$$y_{e'} = 129.4 \text{ mm}$$

$$V_{ee'} = 57.2 \text{ KN}$$

$$S_{ee'} = 143.3 \text{ KN}$$

$$T_{c,e} = 91 \text{ KN}$$

Step 7: Determine the geometry and forces in strut b-b' and d-d''

Strut “b-b'” carries the remaining joint hoop force that was not carried by strut a-a'.

Hence

$$H_{bb'} = V_{sh} - H_{aa'} = 498.0 - 262.7 = 235.3 \text{ KN}$$

The position of node “b” can now be determined

$$y_b = -176 \text{ mm} + (235.3 \text{ KN}) / 2 / 1.41 \text{ KN} / \text{mm} = -92.6 \text{ mm}$$

A similar procedure is conducted for strut d-d'-d''

$$H_{dd''} = 498.0 - 131.34 = 366.6 \text{ KN}$$

The position of node d'' can now be determined

$$y_{d''} = -176 \text{ mm} + (366.6 \text{ KN}) / 2 / 1.41 \text{ KN} / \text{mm} = -46.0 \text{ mm}$$

Nodes on column interior bars are required for equilibrium. There are several unknowns in this strut including the magnitude and position of the column compressive force required for the equilibrium, the level of position of node d'. Several iterations are required to meet the equilibrium conditions and the final result is shown in **Figure IV.1**.

Step 8: Determine the geometry and forces in strut c-c'

This diagonal strut carries the remaining joint shear force that was not taken by the joint hoops.

$$H_{cc'} = V_{jh} - V_{sh} = 913.5 - 498.0 = 415.5 \text{ KN}$$

$$V_{cc'} = 415.5 \times 414 / 356 = 483.2 \text{ KN}$$

$$S_{cc'} = \sqrt{(415.5 \text{ KN})^2 + (483.2 \text{ KN})^2} = 637.2 \text{ KN}$$

Step 9: Calculate the uniaxial compressive stress at the center of the joint. The width of the strut cc' is taken as the nearest distance measured from the joint center of the adjacent struts.

$$W_s = 73.0 \text{ mm}$$

$$f_{c,s} = \frac{S_{cc'}}{d_m b_j} = \frac{637 \times 10^3}{73 \times 250} = 34.9 \text{ MPa} = 0.44 f'_c$$

And

$$v_{jh} = \frac{V_{jh}}{b_j h_c} = 7.6 MPa = 0.095 f'_c$$

Thus

$$\frac{f_{c,s}}{v_{jh}} = 4.6$$