Experimental and analytical study on lightly reinforced concrete frames and beam-column joints

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EXPERIMENTAL AND ANALYTICAL STUDY ON LIGHTLY REINFORCED CONCRETE FRAMES AND BEAM-COLUMN JOINTS

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SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING
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Abstract

Beam-column joints are one of the most critical parts of a reinforced concrete frame structure subjected to seismic loadings but buildings in the non-seismic regions are normally designed to resist gravity load only. Beam-column joints in these buildings pose structural details in contrast to the modern seismic design because the design of such beam-column joints does not follow the seismic design criteria or not satisfy the recommendations by modern seismic design codes. This project investigates the behaviour of lightly reinforced concrete beam-column joints and wide beam-column joints under simulated earthquake excitation-reversed cyclic loadings. The scope of this project includes experimental investigations on the seismic performances of beam-column joints, analytical investigation on either reinforced concrete frames or beam-column joints via a FEM approach, and the parametric study on the beam-column connections.

A literature review of the previous studies focusing on beam-column joints of conventional and wide beam-column connections are presented. Two classical mechanisms representing the behaviour of beam-column joints under reversed cyclic loading are highlighted, i.e. the main strut mechanism and the panel truss mechanism. Different failure modes of a beam-column joint were identified with particular interest in the latest developments in novel models. In addition, three modern seismic design standards for the design of beam-column joints were introduced with concern towards the seismic provisions as well as the recommendations.

Analytical studies on the gravity-load designed reinforced concrete frames were carried with comparing of the test results by other researchers. A gravity-load designed RC frame has the potential lateral load resisting capacity such that the frame could sustain its load-carrying capacity in a low to moderate earthquake. However, strength degradation

and a large magnitude joint shear demand were apparent at a larger deformation level leading to a joint hinge mechanism.

Experimental testing on lightly reinforced concrete interior beam-column joints with a slab and wide beam-column joints under reversed cyclic loading is presented. The specimens represented typical beam-column joint details from the existing low-rise and moderate-rise buildings in Singapore. All conventional specimens experienced severe stiffness degradation and gradual storey shear strength deterioration. It was realized that insufficient transverse reinforcement in the joint panel resulted in a poor performance. The presence of slab increased the storey shear strength but the slab effect on this joint shear strength was much less significant. It was also found that the effective slab width was relative to the flexural strength ratio.

With comparison to the conventional specimens, the wide beam specimens either interior or exterior connections in this study performed well during the test. A larger joint section resulted in the reduction of the nominal joint shear stress therefore joint shear failure could be prevented in these specimens. In a wide beam system, the outside beam portions generate a great concern because bars passing through the outside beam portions lead to a different load transfer mechanism.

Finally, in the parametric study, some of the important experimental information in the recent literature was studied collectively. Parameters that were investigated included: concrete compressive strength, bond index, beam reinforcement ratio, column axial load, slab effect and transverse beams. Interaction between these key parameters and the disparity between ductile and lightly reinforced beam-column joints with respect to each of these key parameters were explored. The key findings based on this limited database study were also compared with relevant requirement in the three seismic design codes.

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List of Symbols

A_g	gross area of column section
A_{scol}	total area of column longitudinal bar
A_{sb}	total area of beam longitudinal bar
A_{sj}	total area of joint reinforcement
A_s	area of tension reinforcement
A'_s	area of compression reinforcement
A_i	frontal area at the i^{th} floor
A_{jh}	total area of effective horizontal joint shear reinforcement
A_{jv}	total area of effective vertical joint shear reinforcement
b_b	beam width
b_w	beam width
b_c	column width
b_j	effective joint width
b_e	effective beam width
C_{c1}, C_{c3}	concrete compression force in beam
C_{c2}, C_{c4}	concrete compression force in column
C_{s1}, C_{s3}	compression force in beam longitudinal bars
C_{s2}, C_{s4}	compression force in column longitudinal bars

C_f	wind force coefficient
d_b	beam bar diameter
d_c	column bar diameter
D_c, D_s	diagonal concrete compression force
E_c	elastical modulus of concrete
E_s	elastical modulus of steel
f	shape factor in Section 4.8
f_{dc}	diagonal joint compression stress
f_{dt}	diagonal joint tension stress
f_t	tension strength of concrete
	diagonal tension stress in Chapter 2
f_c	compression stress of concrete
f'_c	compression strength of concrete
f_u	ultimate strength of steel
f_y	yield strength of non-pressed steel
f_{yh}	yield strength of horizontal joint shear reinforcement
G_f	fracture energy density of concrete
G_c	compression fracture energy
g_k	distributed dead load
h	storey height or vertical distance between column end pins
h_1	distance between the transducers at fixed end interface in beam
h'_1	distance between the transducers at fixed end interface in column
h_b	beam depth

h_c	column depth
h_i'	distance between the left and right transducers in a certain region
h_i	distance between the top and bottom transducers in a certain region
I_e	effective moment of inertia
I_g	moment of inertia based on gross concrete section
k_{th}	theoretical initial stiffness
l	beam span or horizontal distance between beam end pins
l_b'	distance from column face to the center of the beam end pin
l_c'	distance from beam face to the center of the column end pin
l_j	initial diagonal length of a joint
M_{col}	column moment at beam centerline in Sections 2.1.1, 2.1.2.2
M_b	beam moment at column centerline in Sections 2.1.1, 2.1.2.2
M_r	column to beam moment capacity ratio
M_1, M_2	bend moment at beam-joint interfaces in Section 2.3.2
M_3, M_4	bend moment at column-joint interfaces in Section 2.3.2
N_c	column axial load
P_{th}	theoretical load-carrying capacity
P_y	theoretically calculated load-carrying capacity
q_k	distributed live load
S_1	correction factor to account for variations in the wind environment due to topography
S_2	correction factor to account for general ground roughness, building dimension and the height above ground

S_3	correction factor to account for the statistical characteristic of the wind
s_i	gauge length of a certain regain in the beam
s_i'	gauge length of a certain regain in the column
T_1, T_2	tension force in beam longitudinal bar
T_3, T_4	tension force in column longitudinal bar
V	basic wind speed defined as the 3-sec gust with 50 years of a return period, 10 m above ground in an open area with no obstructions
V_b	beam shear force
V_{col}	column shear force
V_{ch}	horizontal joint shear resistance force
V_{cv}	vertical joint shear resistance force
V_{jh}^*	total horizontal design shear force across a joint
V_{jh}	total nominal horizontal shear force across a joint
V_{jv}	total nominal vertical shear force across a joint
V_{jx}	total nominal horizontal shear force in X direction
V_{jz}	total nominal horizontal shear force in Z direction
V_n	nominal shear strength of the joint
W_{ki}	wind load at the floor level
x_i	distance from column face to the center of the region i
y_{th}	theoretical initial yield displacement
y_i	distance from beam face to the center of the region i
θ_{bfe}	beam fixed-end rotation

$\theta_{b,i}$	rotation measurement over a certain region in the beam
θ_{cfe}	column fixed-end rotation
$\theta_{c,i}$	rotation measurement over a certain region in the column
θ_y	yield rotation of beam
$\theta_{\max}$	maximum rotation of beam
$\phi_{c,i}$	average curvature over a certain region in the column
$\phi_{b,i}$	average curvature over a certain region in the beam
δ, δ_j	change of joint diagonal length
$_b \delta_1$	bottom displacement measured at fixed end interface
$_t \delta_1$	top displacement measured at fixed end interface
$_b \delta_i$	bottom displacement measured over a certain region in the beam
$_t \delta_i$	top displacement measured over a certain region in the beam
$_l \delta_1$	left displacement measured at fixed end interface
$_r \delta_1$	right displacement measured at fixed end interface
$_l \delta_i$	left displacement measured over a certain region in the column
$_r \delta_i$	right displacement measured over a certain region in the column
δ_{bf}	beam flexural deformation
δ_{bfe}	deformation due to beam fixed-end rotation
$\delta_{bfl,th}$	theoretical flexural deformation of beam
$\delta_{b,s,th}$	theoretical shear deformation of beam
$\delta_{cfl,th}$	theoretical flexural deformation of column

$\delta_{c,s,th}$	theoretical shear deformation of column
$\Delta T_{c1}, \Delta T_{c3}$	fraction of the total compression force, which transmitted by the beam compression bars by means of bond forces
$\Delta T_{c2}, \Delta T_{c3}$	fraction of the total compression force, which transmitted by the column compression bars by means of bond forces
Δ	applied lateral displacement
$\Delta_{0.75}$	lateral displacement at the peak of loading cycle $0.75P_y$
Δ_{bf}	horizontal displacement due to beam flexural deformation
Δ_{bfe}	horizontal displacement due to beam fixed-end rotation
$\Delta_{b,th}$	theoretical horizontal displacement due to beam deformation
Δ_{cfe}	horizontal displacement due to column fixed-end rotation
$\Delta_{c,th}$	theoretical horizontal displacement due to column deformation
Δ_{cf}	horizontal displacement due to column flexural deformation
Δ_j	horizontal displacement due to joint shear distortion
$\Delta_{j,th}$	theoretical horizontal displacement due to joint shear distortion
Δ_y	yield displacement
$\Delta_{y,th}$	theoretical initial yield displacement
α	over-strength factor
β	factor to reflect the amount of joint reinforcement in Section 8.3.2
ϕ	factor to reflect the reinforcing details in the joint in Section 8.3.2
ϕ	strength reduction factor
γ	constant coefficient based on the joint classification and joint type
	coefficient on joint damage in Section 3.4.2.3
	joint shear stress factor in Section 8.4.1
γ_j	average joint shear distortion
v_u	allowable joint shear stress defined by AIJ

μ_{θ}	rotational ductility factor
μ_{Δ}	displacement ductility factor
τ_{av}	average joint shear stress
ε_y	yield strain of steel
ρ_{top}	beam top reinforcing ratio
ρ_{bot}	beam bottom reinforcing ratio

Chapter 1

Introduction

1.1 Background

Reinforced concrete structures in the areas of low to moderate seismicity are historically designed for resisting gravity loads without regard to any significant lateral forces, this class of structures is referred to as gravity-load-designed (GLD) structures or lightly reinforced concrete (LRC) structures.

In Singapore, a large number of buildings are designed as GLD structures or LRC structures and therefore, they have a little or no transverse reinforcement in the beam-column joint zone. Problems have been encountered when assessing the seismic behaviour of the joints in these buildings. This is due mainly to the absence of a theoretical basis for evaluating the joint behaviour that is supported by comprehensive test data. Modern design codes also have little assistance in assessing these joints because they are developed based on test results of the joints with adequate joint transverse reinforcement.

It is generally accepted that, due to a lack of confinement and ductility, lightly reinforced beam-column joints may be vulnerable when subjected to reversed cyclic loadings. Recent post-earthquake investigations indicate that extensive damage occurs as a result of the excessive shear deformation of the beam-column joints in low and middle-rise non-seismically designed RC frames, and thus leads to the collapse of the buildings.

The behaviour of beam-column joints under reversed cyclic loadings has been studied over the past forty years with much controversial debate and, as an outcome, design guidelines

and design codes have been prepared to assist engineers for their design works. However, most of these studies have been focusing on full ductile details and there is a scarcity of information on the lightly reinforced beam-column joints plus very little research has been done to evaluate the structures in the regions of low to moderate seismicity. There is a need therefore to study the behaviour of lightly reinforced beam-column joints under simulated seismic loadings.

1.2 Seismic Behaviour of Beam-Column Joints and Design Criteria

In order to prevent the beam-column joint failure, current design codes [A2, C1] emphasize that joints must be strong enough to withstand the yielding of the connected beams and columns. This can be achieved by the following principles:

- (1) To make sure that a beam failure occurs earlier than a column failure;
- (2) To ensure the integrity of the joint so that the strength and deformation capacity of the connected members can be developed;
- (3) Strength and stiffness degradation of the joint must be prevented to reduce the increase of lateral displacement.

If a beam-column joint is designed to fulfill these criteria, it should be able to transfer the induced forces and achieve the desired performance.

It is widely accepted that, in a beam-column joint, transferring of the forces coming from the adjunct members is performed via two resisting mechanisms: a diagonal concrete compression strut mechanism and a truss mechanism. Different design approaches are developed with the emphasis on the role of the diagonal concrete compression strut or truss mechanism. Both the ACI 318 [A2] and AIJ [A7] design codes emphasize the diagonal concrete compression strut mechanism and assume that the role of joint transverse

reinforcement is to confine the joint concrete, whereas the NZS 3101 [C1] design code emphasizes the truss mechanism, which is provided by a considerable amount of transverse reinforcement.

1.3 Problem of Beam-Column Joints with Non-Standard Reinforcing Details and Design limits

With reference to the above design criteria and design approaches, beam-column joints in a gravity-load-designed reinforced concrete moment resisting frame may lead to a different behaviour from a seismically designed beam-column joint.

In a non-seismically designed reinforced concrete frame, beam-column joints are not confined by the joint transverse reinforcement, therefore the desired truss mechanism will not develop. As a result joint shear forces can only be resisted by the diagonal concrete compression strut. During an earthquake, the contribution of the concrete to the joint shear strength is gradually reduced and the joint shear failure will occur eventually once the diagonal concrete compression strength is reached. Deficiency of the non-seismically designed beam-column joints will be described in the following chapters.

Generally, wide beam structures are designed for gravity load resisting. However, differences between such kinds of structures with conventional beam structures are evident due to the presence of the wide and shallow beams. A wide beam is wider than the supporting columns and some of the beam longitudinal bars are anchored in the transverse beams rather than anchored in the column core. Comparing with a typical beam-column connection, the wide beam-column joint will lead to a different inter-force transfer mechanism [Q1]. Wide beam moment resisting frames are normally designed in accordance with a standard design philosophy, which is suitable for typical reinforced concrete building designing, and make some of special requirements. However, due to the non-typical characteristics as mentioned as above and characteristics such as column to

beam flexural strength ratio, a lower beam and overall stiffness and bond loss of the column longitudinal bars, the seismic performance of such designed wide beam-column joints is suspected.

Research on the wide beam-column joints has been performed over a twenty year period. Design guidelines and recommendations are highlighted based on the results of these studies. However, due to the insufficient information, the concern of the design of wide beam structural buildings still remains and most design codes restrict the use of wide beam-column joints in lateral loading resisting frames. Meanwhile, some seismic design codes specify some provisions on the use of the wide beam structural system.

Among these design provisions and recommendations, ACI-ASCE 352 [A3] suggests not using of wide beam structures to resist seismic forces. In its recommendations, the wide beam performance is identified as an area needing further research.

The ACI 318 [A2] design code does permit the use of wide beam structures in seismic regions, but restricts the beam width. Note that the restrictions contained in ACI 318 [A2] are actually based on the experience of the code writer rather than on research.

The NZS 3101 [C1] design code allows the use of wide beam structures in seismic regions, but requires that the width of the beam is at most twice the width of the column.

1.4 Objective and Scope of the Study

The objective of this study is to evaluate the inelastic performance of the non-ductile moment resisting RC frames designed conforming to the BS 8110 [B3] design code. An experimental and analytical study of are to be undertaken with this concerns. The results will be useful in identifying the weakpoints of this kind of structure and establishing the hierarchy of failures.

This study looks into typical low-rise reinforced concrete moment resisting frame structures with non-ductile detailing at the beam-column joints, which are recognized as one of the most critical part of a structure under reversed cyclic loadings. A lightly reinforced beam-column joint is fundamentally characterized by the absence of joint transverse reinforcement within the joint panel. The results and conclusions obtained from this study are supposed to serve the purpose of estimating the overall performance of the structures subjected to seismic loadings. As far as this project is concerned, the objective is to understand the fundamental mechanism of lightly reinforced beam-column joints and how various parameters affect their behaviour under reversed cyclic loadings. In addition, the seismic behaviour of wide beam-column joints is studied to understand the potential of a wide beam structural system as a lateral load resisting structure.

The scope of this project covers three major parts. Firstly, the pre-tested gravity-load-design reinforced concrete moment resisting frames were analyzed using a finite element analysis approach. Different failure models of a beam-column joint were identified with a particular interest in the latest developments of novel models.

Secondly, experimental studies including five full-scaled lightly reinforced interior beam-column joints and three interior and three exterior wide beam-column joints were conducted under reversed cyclic loadings with the experimental programs being carried out at Nanyang Technological University. The research program summarized in this thesis is aimed at investigating the performance of conventional and wide beam-column joints subjected to earthquake actions, which are not addressed in Singapore.

Finally, a parametric study was conducted with an intention to identify the influence of variables and to understand the interactions among various parameters and, based on these works, design recommendations and simple assessment approaches were proposed.

This research consists of the following experimental and analytical components:

- (1) Identifying structural details and features of non-seismically designed reinforced concrete moment resisting frame buildings.
- (2) Evaluating seismic performance of non-seismically designed reinforced concrete moment resisting frame buildings.
- (3) Conducting a series of experiments to study the seismic behaviour of beam-column joints with substandard detailing.
- (4) Undertaking a parametric study to provide additional information for understanding of the variable influences of beam-column joints in the reinforced concrete frames designed with out seismic provisions.

The long-term purpose of this research program is to provide guidelines for practicing structural engineers in evaluating existing reinforced concrete moment resisting frame structures, mitigating seismic hazards, and reducing the risk level to building occupants and owners in Singapore.

1.5 Organization of the Thesis

The thesis is organized into nine chapters. Chapter 1 provides an introduction to the background and the scope of this research. Chapter 2 presents a literature review of the previous analytical and experimental researches on the subject of non-ductile designed beam-column joints and wide beam-column joints. In Chapter 3, two gravity-load-designed and detailed reinforced concrete moment resisting frame structures are analyzed by using non-linear finite element analysis. The seismic assessment of these buildings is also described in this chapter. Chapter 4 describes the testing methodology and specimens design. Chapter 5 presents the test results of the non-ductile beam-column joint specimens associated with the analytical results using 3D finite element approach. Chapters 6 and 7 discuss the test results of interior and exterior wide beam-column joint specimens, respectively. Extensive parametric studies of normal and wide beam-column joints are

presented in Chapter 8. Finally, a summary of the studies and recommendations are given in Chapter 9.

Chapter 2

Literature Review

2.1 Gravity-Load-Designed (GLD) Beam-Column Joints under Seismic Conditions

2.1.1 Characteristics of a Gravity-Load-Designed Reinforced Concrete Moment Resisting Frame and Beam-Column Joint

Before employment of seismic design principles in the mid of 1970's, reinforced concrete moment resisting frames were designed to resist gravity loads and wind loads (e.g., according to ACI 318 [A2]) with a corresponding bending moment pattern as shown in **Figure 2-1(a)**. It can be seen that hogging moments occur in the top surface of beam ends frame into the joints, thus beam top bars are subjected to tension stress at beam-column interfaces. Problems of bond deterioration may not happen under this situation. The design of such kind of buildings was based on the considerations of elastic behaviour of the structural system as well as components with requiring of the adequate strength to resist the gravity loads. Lateral loads with exception of wind load, however, were ignored or simply specified as a function of the dead weight of the building, in which the resulting bending moment pattern are shown in **Figure 2-1(b)**. It is clear that only one beam end moment becomes the hogging moment while the other end of beam has a sagging moment, and the column end moments also have opposite signs across the joint.

The majority of existing old buildings found over the world designed conforming to the early design codes without seismic provisions can be taken as the gravity-load-designed (GLD) buildings or lightly reinforced concrete (LRC) buildings. Up to now, the

gravity-load-design approach is still a typical design practice in the low to moderate seismic regions such as Singapore and Malaysia. In Singapore, buildings are designed in accordance with the provisions of BS8110 [B3], which do not specify any seismic detailing provisions.

Figure 2-1 Bending moment diagrams of gravity-load and lateral-load resisting frames

The shear stresses and moments diagram of a beam-column joint is given in **Figure 2-2**. The existence of shear forces in the joint region is evident from the moment gradient along both the beams and columns and also a large magnitude of joint shear forces is expected because the gradient of the moment diagrams is much steeper through the joint than that in the middle portion of the members. The horizontal shear forces at the joint equal to the sum of the column moments divided by the beam depth, while the vertical shear forces equal to the sum of the beam moments divided by the column depth. Corresponding joint shear stresses, therefore, are the sum of these moments divided by the effective joint volume, which is the multiplication of the beam depth, column depth and effective joint width. Both ACI 318 [A2] and NZS 3101 [C1] have similar definitions about the effective joint width.

Through a review of earlier versions of design codes and related design manuals, the general characteristics of GLD reinforced concrete moment resisting frames have been summarized by many researchers [E4, B6, B7, M4, A9, P1, K4 and S2]. Here it is outlined as follows considering some reinforcing features shown in **Figure 2-3**.

- (1) A strong column-weak configuration is not a design criterion for the gravity-load-design approach.
- (2) Within a joint region, the amount of beam top bars is normally twice of that on the bottom.
- (3) Beam bottom bars in the vicinity of the interior joint region are either discontinuous or with a very short embedment length into the joint core.
- (4) Limited or no transverse reinforcement is provided within the joint core.
- (5) A low reinforcing ratio around 2% is provided in columns. In addition, column longitudinal bars are lapped immediately above the floor level in the zone of the maximum bending moment, creating a soft-storey or column side-sway mechanism.
- (6) Shear forces are not generally to be considered in the column design resulting in widely spaced column transverse reinforcement. Both column ends as well as beam ends are slightly confined, which result in an adequate confinement effect in potential hinge regions.

2.1.2 Summary of Deficiencies of Gravity-Load-Designed Beam-Column Joints under an Earthquake Attack

The deficiency of non-ductile construction became apparent in the aftermath of the 1971 San Fernando earthquake. Recently, it has been recognized that safety and vulnerability of GLD reinforced concrete buildings, in which ductile detailing had not been provided explicitly in the design process, should be a concern. The performance of the GLD reinforced concrete buildings in an earthquake event is being under move investigation due to the poor reinforcing details and the absence of any capacity design principles in their

design process. It is believed that the deficiency in describing the seismic performance of existing GLD buildings is mainly attributed to lack in consideration of ductility rather than an inadequate lateral strength problem. Because, even though the GLD structures are designed without due considerations to lateral loads, they may still possess an inherent lateral capacity, which may be adequate to resist a minor to moderate earthquake. However, the deficient detailing of members, in which a significant lack of ductility consideration at both local and global levels is evident, can lead to questionable behaviour of the buildings during severe seismic activity [B6].

Figure 2-2 Shear stresses and moment diagrams of an isolated interior beam-column joint

2.1.2.1 Performance of Beam Bottom Bars in the Joint Core

Referring to the moment diagram (**Figure 2-1(a)**), only the negative moment (or hogging moment) occurs at top surface of the beam framing into the joint within the joint region. Thus, only a certain length would be needed of the beam bottom bars entering into the column support, meeting the requirement for the bars to fully develop their strength. Therefore, the beam bottom bars are usually cut off in the joint core.

In the case of an earthquake attack, however, the beam bottom bars will subject to a sustained reversed cyclic process in which the bars are applied by tension and compression

forces or being pulled out and then pushed in alternately. For this situation, the anchorage length of beam bottom bars within the column support would be no longer enough to develop their strength. The termination of positive bars within the beam-column joint will result in these sections being not able to reach their yielding moment, because the bond between the bars and the joint core concrete is inadequate to develop a yield force in the steel [K4]. In the inelastic range and particularly under cyclic loadings, a progressive loss of the bond between the bars and concrete is expected and as a result, bar slipping through the joint core cannot be avoided and consequently beam cracks at the column faces will open wide resulting in beam fixed-end rotation. It was reported by Ma *et al* [M1] that cracking of a beam in the inelastic range is important to the global behaviour of the frame because of the large contribution made to the tip deflection. Beres *et al* [B1] have reported a similar situation in their non-ductile beam-column joint tests.

Figure 2-3 Reinforcing features of GLD reinforced concrete frame

2.1.2.2 Role of Joint Transverse Reinforcement

Lack of joint transverse reinforcement would be another weak point of GLD reinforced concrete structures under seismic actions. Such a reinforcing technique is accepted based on the understanding that under gravity loads the shear forces induced in the joint region

will be low enough that it can be carried by the concrete and longitudinal bars. Contrary to the GLD principle, the joint core in a seismic designed frame is considered as an area undergoing large shear forces. Under seismic conditions, the diagonal tension stress f_t within the joint core develops concurrently with the diagonal compression stress f_c as shown in **Figure 2-4**. When large enough, the tension stresses will cause diagonal cracking of the diagonal concrete compression strut, and then reduce its compression strength.

The role of joint transverse reinforcement is described in different shear resisting mechanisms of joints. In ACI 318 [A2], joint transverse reinforcement is considered to provide confinement to the joint core. Under reversed cyclic loadings the transverse reinforcement will integrate the core and thus the joint will sustain its capacity even under large joint deformation. On the other hand, in NZS 3101 [C1], joint transverse reinforcement is considered to carry joint shear forces. The transverse reinforcement, longitudinal bars in the column and numerous concrete struts together form a truss mechanism to transfer joint shear forces. Although different understandings do exist, a lack of joint transverse reinforcement will eventually lead to the joint shear failure. This can also lead to a non-ductile failure once the shear capacity of the concrete has been exceeded [K4].

Figure 2-4 Interior joint under seismic actions and joint shear failure

2.1.2.3 Deficiency of Column Longitudinal Bars

In most GLD buildings, a strong column-weak beam configuration is not required. Instead, beams are always designed stronger than columns. The column reinforcing ratio is dominated by the column compression strength; furthermore, shear forces in the column design are always neglected, leading to a large spacing of column transverse reinforcement. As part of the typical construction techniques, column longitudinal bars are lapped just above the floor level in a potential hinge region. Reinforcing details for the columns in a GLD structure are opposite to those required in the seismic designed practice. Under seismic conditions, column ends are considered as regions that will undergo a large magnitude of reversed moments combined with large axial forces. Hence, column hinges are expected in these regions. As early as 1982, Paulay [P6] had pointed out that when the lap splices were placed in plastic hinge regions, a large amount of transverse reinforcement must be provided to ensure that the longitudinal reinforcement forces could be transferred satisfactorily within the lap length. If clamping forces could not be applied, yielding of the longitudinal reinforcement was found to be restricted to a small length at the end of the laps, leading to an undesirable concentration of curvature there and resulting in extremely large steel strains, premature bulking of bars, and in some cases the fracture of the longitudinal bars, that is, the yielding did not spread along the column.

2.1.2.4 Role of Beam and Column Transverse Reinforcement

The role of beam and column transverse reinforcement is to aid the development of the longitudinal bars strength, confine the compressed concrete, prevent premature bulking of the longitudinal bars, and to prevent the shear failure of members. To prevent the premature bulking of the compressed longitudinal bars, Bresler [B2] recommended that within a length equal to one-sixth of the column height, at each end of the column the tie spacing should not exceed $6d_b$. A similar situation is expected in the beams when the building is subjected to an earthquake attack. It is implied that transverse reinforcement in the GLD

columns will not be able to apply clamping forces across the potential longitudinal splitting cracks and therefore disable to provide a shear friction mechanism to transfer the forces from one spliced column bar to another. Lightly confined beam ends do not possess a large deformation capacity, and the premature bulking of the beam longitudinal bars cannot be prevented due to the wide spacing of beam transverse reinforcement.

2.1.3 Previous Investigations on Gravity-Load-Designed Buildings

Extensive investigations on the behaviour of GLD buildings under seismic conditions started at the 1990s. The information was early reported by Pessiki *et al.* (1990) [P8] and since then, a lot of researchers begun working on this topic and as a result presented much valuable information such as Hoffmann *et al.* [K4], Bracci *et al.* in 1995 [B6, B7], Mander *et al.* in 1995 [A9], and El-Attar *et al.* in 1997 [E4]. Their research scope includes the seismic response of gravity-load-designed RC buildings; failure modes; deficient detailing on the overall performance of the structure frame; retrofit techniques and improving strategies. Based on these investigations, it is approved that:

- (1) The reinforced concrete buildings designed for gravity loads only have some inherent capacities to resist earthquake actions, especially for the moderate earthquakes. However, a large inelastic deformation with substantial inter-storey drifts during a moderate earthquake cannot be avoided.
- (2) Due to a weak column-strong beam solution in the gravity-load-design and the inadequate ductility in the column sections, the system failure mode is dominated by a relatively brittle soft-storey or column side-sway collapse mechanism. At the same moment, stiffness is found to be reduced in the structure and individual members.
- (3) Normally, structures will experience a significant reduction in lateral stiffness after the first few cycles of a moderate earthquake. This is mainly due to the lack of ductility by poor reinforcing details and the pulling out of the discontinuous positive beam bars in the joint core. After a severe earthquake, serious column damage is expected and their

effective stiffness can deteriorate to as low as that of 20% of the gross section properties [B6]. However, structural components can reach their flexural strength and still sustain their gravity loads during the large deformation cycles.

Evaluation of the retrofit techniques and strategies of reinforcing improvement were summarized by Kunnath *et al.* [K4]. They reported that:

- (1) The provision of beam bottom bar continuity significantly reduced the beam damage by reducing the amount of hinge rotation. However, it did transfer the majority of the damage to the columns and resulted in an even greater number of joint failures. This obviously magnified the soft-storey effect of the structure. Due to a non-ductile failure mechanism that remained, it was evident that the provision of beam bottom bar continuity alone would be detrimental to structural performance.
- (2) Joint shear capacity could be restored by the provision of transverse reinforcement, resulting in a tendency to shift damage from columns into beams. As a result, a beam side-sway mechanism could be expected. However, due to a significant lower capacity of the beam, storey drift was still at an unfavourable level.
- (3) Confinement in the plastic hinge regions could increase members' rotational capacity; however, if independent of other enhancements, additional confinement does not affect the overall performance demand of the structure. To achieve the best benefits a combination of the three detailing strategies should be adopted [K4].

2.2 Recent Investigations on Non-Ductile Beam-Column Joints

It is well recognized that the survival of a structure under an earthquake attack depends mainly on its ability to absorb and dissipate energy during the post-elastic response. According to more recent researches, the excessive shear deformation in the joint region and the reinforcing bars slipping due to the bond deterioration, contribute significantly to the ultimate damage of structures. Under moderate to strong earthquakes, the beam-column

joint region suffers severe stiffness degradation and strength deterioration. This is especially true for GLD assemblages or lightly reinforced concrete beam-column connections. Therefore, it is important to investigate the behaviour of non-ductile beam-column joints subjected to lateral cyclic forces.

2.2.1 Research Conducted by Blaikie

A theoretical model is presented which endeavors to explain how the joint shear forces to be applied, and their effect on the stress induced in the beam and column flexural bars [B4]. It was proposed that the presence of joint transverse reinforcement will significantly affect the shear force resisting and the performance of joint panel as well as the adjacent members. The adequate joint transverse reinforcement will provide a full horizontal restraining force to complete flexural cracks open at the beam-joint interfaces, whereas, the light joint transverse reinforcement will only provide some of the required restraining force to prevent the expansion of the joint. If the joints are designed without adequate transverse reinforcement, however, the beam and column flexural bars within the joint will have a dual function acting as both the flexural and shear reinforcement.

According to the proposed model, the flexural steel will be therefore more highly stressed within the joint than that in other portions of the beam or in the column [B4]. A joint hinge mechanism can be expected as the yielding in the adjacent beams or columns being limited.

To assist developing the theoretical model, interior and exterior beam-column specimens without joint transverse reinforcement and applied by zero to low axial load were tested. Results from these tests generally showed good agreement with the behaviour predicted by the theoretical model. In particular, the theoretical model correctly predicted that the interior beam-column joint specimen would develop only 70% of the capacity indicated by the beam and column flexural strengths.

It was suggested by the author that when assessing existing frames without transverse reinforcement in the joints, the effects of ductility on shear strength of the concrete component of the beam need not be considered. However, the effect of the beam end moments being limited by the joint strength and the effect of joint strength degradation will need to be considered.

2.2.2 Research Conducted by Pessiki *et al.*

In the research conducted by Pessiki *et al* [P8], tests were performed to study the seismic resistance of beam-column joints and column splice details commonly used in lightly reinforced concrete frame construction.

The tests were made on full-scale beam-column connections, loaded in a manner causing a combined axial force and reversed lateral load. Six specimens were constructed to study the behaviour of column splice and beam-column joint. Variables in the tests included spliced and un-spliced column bars, column bar arrangement, and varying amount of joint transverse reinforcement. Four specimens were prepared to study the pullout resistance of embedded positive beam bars. Variables included embedded bar sizes, column axial forces, and presence of transverse beams.

It was found from the tests that the column splices performed better than that was expected. Severe damage to the top columns, splitting cracks and loss of the concrete cover were always limited in a region below the first column transverse reinforcement. All the specimens showed little abilities to maintain peak resistance for the reversed cyclic loadings. For those specimens contained more joint transverse reinforcement than which would typically be found in lightly reinforced structures, the peak resistance could be maintained with continued cycling at a larger drift level. The specimens obtained the maximum joint shear stress of about $1.9\sqrt{f'_c}$ (f'_c unit in MPa), independent of the arrangement of column bars or the presence of minimum joint transverse reinforcement.

Failure of each specimen with embedded beam bottom bars initiated by pullout of the embedded bars from the beam-column joint. The maximum joint shear stress reached in each specimen was about $1.5\sqrt{f'_c}$ (f'_c unit in MPa), which was below the value that caused the joint failure in the specimens with continuous beam bar. The pullout resistance was largely independent of bar sizes and column axial loads, and there might be small increase in the strength caused by transverse confinement.

2.2.3 Research Conducted by Hakuto *et al.*

In Hakuto's research [H1], six one-way interior beam-column joints and two exterior beam-column joints were constructed and tested. These test specimens were selected from an existing seven-storey reinforced concrete building that was designed conforming to the older design codes in New Zealand and the United States.

When the specimens were loaded in the post-elastic range, the available shear strength of the interior beam-column joints without transverse reinforcement decreased with the increase in the imposed ductility of the adjacent beam plastic hinge regions. The test results indicated that the seismic performance of the interior beam-column joints without transverse reinforcement in the joint core would be poor in a severe earthquake if the nominal horizontal joint shear stress exceeds approximately $0.61\sqrt{f'_c}$ (f'_c unit in MPa). The effect of bond conditions on the seismic performance of the joints was also investigated. However, the influence of different conditions was found to be of little significance.

For the exterior beam-column joints, the seismic performance was significantly influenced by the reinforcing configuration, particularly the direction of the hook at the ends of the beam longitudinal bars anchored inside or outside the joint core. When the beam bar hook was not bent into the joint core, it was not able to develop the diagonal concrete compression strut mechanism within the joint along a corner to corner diagonal.

Based on the test results, a principal tension model was proposed by Hakuto *et al* [H1]. This model related the joint shear stress level to the joint ductility factor, giving a simple and clear way to assess the behaviour of interior beam-column joints without transverse reinforcement. However, it is noted that all the specimens were tested without column axial loads, the proposed model may be inappropriate to assess the behaviour of beam-column joints under column axial loads.

2.2.4 Research Conducted by Aizhen Liu *et al*.

The non-ductile interior beam-column joints as well as exterior beam-column joints were tested by Liu *et al*. [L2]. The geometries and reinforcing details of Liu's test specimens were identical to the specimens tested by Hakuto *et al* [H1]. The primary difference is that, the plain round bars were used in Liu's experiments, whereas the deformed bars were adopted by Hakuto *et al* [H1].

The test results on the interior beam-column joints indicated that the utilization of plain round longitudinal bars resulted in lower structural stiffness and lower strength, but they could enhance the joint shear resistance associated with the diagonal concrete compression strut. In this experiment, the specimens were tested under a column axial load of $0.12f'_c$ (f'_c unit in MPa). This column axial load caused a reduction in the load-carrying capacity of the specimens, and also caused a reduction in the displacement capacity and the stiffness to be reduced as well. This was because the column axial load may induce the buckling of the column longitudinal bars when the column transverse reinforcement is inadequate.

A total of four exterior specimens were tested in this study with concerns of different beam bar anchorage details. The first two specimens had the beam bar hooks bent away from the joint core and the other two specimens had the beam bar hooks bent into the joint core. All the four test specimens were designed with limit joint transverse reinforcement. In general,

the seismic behaviour of the test specimens was controlled by the premature concrete tension failure and due mainly to the severe bond degradation occurring along the beam longitudinal bars within the joint core, the tension forces in the beam bars was mainly transmitted by the diagonal concrete compression strut. Therefore, the concrete tension cracking failure initiated by the beam bar hooks was induced leading to an increased demand for the joint transverse reinforcement to prevent such a failure. However, the concrete tension cracking failure could not be avoided due to the limited amount of joint transverse reinforcement, and as a consequence the diagonal concrete compression strut mechanism was diminished. It also reported that different beam bar hook details in the exterior columns had significant influences upon the performance of the test specimens. When the beam bar hook was bent into the joint core, the obtained storey shear strength was about 75% of the theoretical strength, whereas, if the beam bar hook was bent away from the joint core, the attained storey shear strength was only 55% of the theoretical strength.

Comparing these results on the exterior specimens with those on the interior specimens, it was found that the adverse effect of the use of plain bars on the structural stiffness and strength was more severe for the exterior beam-column specimens than that for the interior beam-column specimens. For interior beam-column joints, the use of plain round bars resulted in the strength and stiffness being reduced by 15% and 23%, respectively, compared to the results of Hakuto's specimens with deformed bars. For exterior beam-column joints, the attained stiffness of Liu's specimens was only 50% of the corresponding Hakuto's specimens.

2.2.5 Research Conducted by Wu *et al.*

In Wu's research [W1], four interior beam-column joints were tested. These specimens were designed to BS8110 [B3] which did not specify any seismic provisions. The main variables of these specimens were column to beam flexural strength ratio, joint transverse reinforcement, and reinforcing details of beam longitudinal bars. These specimens were

picked out from a reinforced concrete wide-column moment resisting frame, which is a typical design and construction practice in Singapore.

Beam-wide column joints studied in this research have been proved to be the critical parts even when the wide-column frames were experiencing a low intensity earthquake. It was found that due to the particular arrangement of column bar lap splices and the absence of joint transverse reinforcement, the lap splice failure was found to occur accompanied by the joint shear failure. Severe strength and stiffness degradation were observed with the joint shear failure which was found to be the major failure mode of the joint. The test specimens could finally reach limited displacement ductility levels.

The joint transverse reinforcement played an important role of improving the joint behaviour. Due to the presence of joint transverse reinforcement, the strength of the specimens could be maintained when the specimens were subjected to a large magnitude of deformation demands. As a result, the joint core became more flexible and contributed more to the total horizontal displacement of the specimen. However, it was found in the tests that the modification of reinforcing details had no obvious effect on improving bond conditions. It was recommended by Wu Y.M. that the benefit of joint transverse reinforcement can be achieved only when the joint shear stresses are less than $0.25f'_c$. If the joint shear stresses are greater than $0.25f'_c$, the joint transverse reinforcement will have no influence on the joint behaviour.

Based on the analytical investigations conducted in this research, it was found that when $N_c / A_g f'_c < 0.4$ the column axial force was beneficial to the joint, whereas when the column axial force was larger than $0.4A_g f'_c$ the column axial loads became detrimental to the joint. Therefore, it was recommended that the beam-column joint behaviour should be evaluated when the column axial force is limited below $0.3A_g f'_c$.

2.3 Design of Beam-Column Joints in a Reinforced Concrete

Moment Resisting Frame

In a seismic designed frame building, joints are taken as the strongest components so that only the strength and ductility of the joints need to be checked. Following the seismic design criteria, the strength of beam-column joints is ensured by limiting its nominal shear stress and constraining the significant shear deformation, furthermore, the bond failure is prevented by limiting of bond indexes.

2.3.1 Shear Resisting Mechanism in an Interior Beam-Column Joint

External forces from beams and columns must be safely resisted within the joint core. Two principal mechanisms of shear resistance have been postulated [P9] and further examined [P10]. **Figure 2-5** represents the shear resisting mechanisms in an interior joint.

Figure 2-5 Shear resistance mechanisms in an interior joint (after Park)

2.3.1.1 Diagonal Concrete Compression Strut Mechanism

The first mechanism consists of the contribution of concrete compression and the shear forces around the joint. In **Figure 2-5 (a)**, C_{c1} and C_{c2} are the stress resultants of the

corresponding concrete compression stress blocks at the upper-right corner. It is assumed that the shear forces V_b and V_{col} are transferred into the joint via the respective concrete flexural compression zones. Also shown at the upper-right corner, ΔT_{c1} and ΔT_{c2} denote the fractions of the total compression forces, which are transmitted by the beam and column compression bars by means of bond forces. These forces are combined into a single diagonal compression force D_c which is assumed to incline at an angle and close to that of the potential diagonal failure plane. A similarly situation occurs at a lower-left corner, where the opposed diagonal compression force D_c forms a compression strut. If the compression strength of the strut is not excessive, the mechanism is effective. The joint strength in this mechanism, sustaining the diagonal compression force D_c , is sometimes referred to as the “shear carried by the concrete”. V_{ch} and V_{cv} are used to represent the horizontal and vertical joint shear resistance, respectively and can be expressed as:

$$V_{ch} = D_c \cos \alpha \quad (2-1)$$

$$V_{cv} = D_c \sin \alpha \quad (2-2)$$

2.3.1.2 Truss Mechanism

Since a fraction of the total bond force ΔT_c has been combined into the diagonal compression force D_c , the remainder of the total bond force, which is introduced to the joint core in terms of shear flows, can be expressed as:

$$V_s = T + C'_s - \Delta T_c \quad (2-3)$$

where, V_s is the horizontal or vertical force transferred by the bond forces from the beam or column longitudinal bars, while $T + C'_s$ represents the total bond force to be transmitted

to the joint (see **Figure 2-4**). Primarily, V_s is assumed to be transmitted to the core concrete mainly outside the diagonal concrete compression zone of **Figure 2-5(a)** and will generate a total diagonal compression field D_s . A truss mechanism generating this compression field action involves horizontal bars, vertical bars, and diagonal concrete struts. If adequate lateral resisting forces at the boundaries are provided through the existence of joint transverse reinforcement, the diagonal compression field D_s can be sustained to resist the shear forces even when extensive cracks are developed in the joint core. Therefore, the truss mechanism consists essentially of the contribution of the vertical and horizontal bars within the joint core.

2.3.2 Shear Resisting Mechanism in an Exterior Beam-Column Joint

It should be noted that the aforementioned joint shear mechanism and bond mechanism are developed in interior beam-column joints. In exterior beam-column joints, since there is only one beam framed into the joint, the horizontal joint shear force is therefore expressed as:

$$V_{jh} = T - V_{col} \quad (2-4)$$

This shear force can also be transferred into the joint core by the diagonal concrete compression strut mechanism as well as the truss mechanism. A diagonal concrete compression strut will be developed between the bend of the hooked beam top bars and the concrete compression stress block at the opposite corner of the joint. Therefore, the diagonal compression force D_c must be balanced by the anchorage forces at the hook of the beam longitudinal bars and the compression forces in both the horizontal and vertical directions at the opposite corner of the joint.

In an exterior beam-column joint, the anchorage detail of beam longitudinal bars and joint transverse reinforcement play important roles in transferring of forces across the joint core. At an early stage, tension forces in the beam bars will be transferred into concrete by bond stresses and bearing stresses at the hook and after the development of beam hinges, bond deterioration occurs. As a result, the bend hook effect on the diagonal concrete compression strut mechanism is enhanced due to the slippage of beam bars. When the beam bars are subjected to a further tension action, the beam tension bars tend to straighten resulting in spalling off of the concrete cover and then loss of protection against the buckling of the column longitudinal bars. At this moment, concrete tension cracking along the beam hooks occurs.

In order to prevent concrete tension cracking, adequate joint transverse reinforcement is required in the design of exterior joints. Therefore, the aforementioned diagonal concrete compression strut between the beam bar hook and joint corner can be achieved resulting in the transition of beam tension forces to those in the diagonal concrete compression strut and truss mechanisms. In addition, joint transverse reinforcement can restrain the opening action of the hook of the beam longitudinal bars and prevent the buckling of the column longitudinal bars.

2.3.3 Definition of Joint Shear Demand

When a building is subjected to combined gravity and lateral loads, the beam and the column end moments have opposite signs across the joint as shown in **Figure 2-2**. According to the elastic analysis, the inflected points of the bending moment diagram are assumed to be located at the mid-height of the column and the mid-span of the beam. Based on this assumption, the statics diagram regarding an isolated interior beam-column connection can be illustrated by **Figure 2-6**. Hence, the horizontal joint shear force V_{jh} can be developed (**Equation 2-5**).

$$V_{jh} = T_1 + C_{s1} + C_{c1} - V_{col} = T_2 + C_{s3} + C_{c3} - V_{col} \quad (2-5)$$

Similarly, the vertical joint shear force V_{jv} is given by **Equation 2-6**

$$V_{jv} = T_3 + C_{s4} + C_{c4} - V_b = T_4 + C_{s2} + C_{c2} - V_b \quad (2-6)$$

Among these notations, T_1 and T_2 are the tension forces in the beam longitudinal bars, C_{s1} and C_{s3} are the compression forces in the beam longitudinal bars and C_{c1} and C_{c3} are the concrete compression forces in the beams. Along the column-joint interfaces, T_3 and T_4 are the tension forces in the column longitudinal bars and C_{s2} and C_{s4} are the compression forces in the column longitudinal bars, C_{c2} and C_{c4} are the concrete compression forces in the columns. It should be noted that, **Equation 2-5** implies a hypothesis that the bond forces along the beam longitudinal bars are sufficient to anchor beam bars, resulting in one beam end in tension and another end in compression. However, the equations for calculation of the horizontal or vertical joint shear force are actually identical with either a perfect or poor bond condition.

Figure 2-6 Definition of joint shear forces

Taking into account of equilibrium of the flexural forces at the right-hand side of the beam-joint interface, **Equation 2-7** can be obtained:

$$T_2 = C_{s1} + C_{c1} \quad (2-7)$$

Substituting **Equation 2-7** into **Equation 2-5**, it can be re-written as:

$$V_{jh} = T_1 + T_2 - V_c \quad (2-8)$$

In the design practices, the tension forces T_1 to T_4 are difficult to be evaluated and the horizontal and vertical joint shear forces are simply determined according to the statics diagrams. In that case, the horizontal joint shear forces are equal to the sum of the column moments divided by the beam depth, while the vertical joint shear forces are equal to the sum of the beam moments divided by the column depth. Consequently, joint shear stresses are obtained by the horizontal joint shear forces divided by the effective joint width multiplied by the column depth. A definition of effective joint width in different design codes is summarized in **Table 2-1**.

Table 2-1 Definition of effective joint width in different design codes

Design Code	Definition
ACI 318-99	$b_j = b_b + h_c$ $b_j = b_c$ whichever is smaller
NZS 3101-1995	$\min(b_j = b_b + 0.5h_c, b_j = b_c)$ when $b_c > b_b$ $\min(b_j = b_b + 0.5h_c, b_j = b_b)$ when $b_c < b_b$
AIJ-1994	The average of the b_b and b_c

The intensity of the vertical joint shear force can be estimated by:

$$V_{jv} = (h_b/h_c) V_{jh} \quad (2-9)$$

where, V_{jh} and V_{jv} stand for nominal horizontal and vertical joint shear forces, h_c is the column depth, and h_b is the beam depth.

2.3.4 Design Provisions by Different Design Codes

Over the past 30 years, extensive experimentations and analytical research have been conducted around the world by many researchers to investigate the behaviour of beam-column joints subject to earthquake attacks. Based on this research, design codes with seismic provisions have been developed. However, they still cannot describe quantitatively the mechanical behaviour of a beam-column joint under earthquake actions because of the complexity of the force transfer mechanisms and material's non-linearity. Up to now, three major seismic design codes, which are: the ACI 318 [A2] design code, the NZS 3101 [C1] design code and the AIJ [A7] design code are accepted by many countries as the guidelines for the beam-column joint design.

2.3.4.1 ACI 318-99 Design Code Requirements

To achieve a “strong column-weak beam” design criteria, the ratio of the column to beam flexural strength must be greater than 1.4 [A2] or 1.2 [C1]. When calculating a beam moment capacity, an over strength factor $\alpha = 1.25$ must be included to account for the actual yield stress of a typical reinforcing bar, which is commonly 10%-25% higher than the nominal value, and for the strain hardening effect after the beam longitudinal bar's yielded.

In ACI 318 [A2], the joint panel is taken as a part of the column. Due to the steeper moment gradient between the opposite sides of the joint, this critical region must be properly designed and detailed to resist a large magnitude of shear forces. The maximum horizontal shear force is approached when the flexural hinging mechanism is formed in the adjacent beam members. In the ACI 318 [A2] design code, the induced joint shear force is

considered to be primarily carried by the diagonal concrete compression strut and column longitudinal bars.

In order to prevent the excessive shear deformation of the joint core, the joint shear force is limited in terms of joint shear stresses, which is a function of “ $\sqrt{f'_c}$ ”. The following equation should be satisfied:

$$\phi V_n \geq V_x \quad (2-10)$$

where $\phi = 0.85$ and V_n is the nominal shear strength of the joint as expressed below:

$$V_n = 0.083\gamma\sqrt{f'_c}b_jh \quad (2-11)$$

where b_j is the effective joint width and h is the thickness of the column in the direction of the loading being considered.

The constant coefficient “ γ ” is given in **Table 2-2**, which was based on the joint classification and joint type. Since the joint is treated as a part of the column, the vertical shear stresses in the joint core are ignored.

Table 2-2 Value of γ for beam-column joint design (by ACI-ASCE 352R [A3])

Joint type	Joint Classification		
	Interior joint	Exterior joint	Corner joint
1	1.7	1.25	1.0
2	1.25	1.0	0.85

f'_c : unit in MPa

The role of transverse reinforcement is expected to provide adequate confinement to the joint during an anticipated earthquake attack and displacement demands rather than carry the fraction of the induced joint shear forces. In the ACI 318 [A2] design code, the

significant confining effect of transverse beams is emphasized resulting in the relaxation of joint transverse reinforcement. It recommends that, when beams are framed into all four faces of the joint, the total transverse reinforcement within the joint can be reduced by 50%.

Much research has shown that straight beam and column bars may slip within the joint during a series of large reversed moments, and the bond stresses on these straight bars may be very large. To limit bond forces and to prevent the slippage of the beam and column bars through the connection, the ratio of the effective embedment length to the beam as well as column bar diameters passing through the joint is restricted to 20 by ACI-ASCE 352 [A3]. It should be noted that to reduce the bond stresses to a value lower enough and to prevent the bar slippage under large reversed loads, a very large connection would be required.

2.3.4.2 NZS 3101-95 Design Code Requirements

In NZS 3101 [C1], both the diagonal concrete compression strut mechanism and truss mechanism are considered to be effective in resisting joint shear forces. However, the first mechanism is only initiated at the early stage of response before the beam yielding starts to penetrate from adjacent beam-column interfaces into the joint. After several significant cycles of loading, both the horizontal and vertical joint shear forces will have to be resisted through a bond mechanism. It is conservatively assumed at this point that the diagonal concrete compression strut mechanism is no longer activated. The contribution of the diagonal concrete strut mechanism is ignored unless a significant column axial load can be expected.

It is proposed that the column axial load contribute to the diagonal concrete compression strut by increasing its steepness towards the vertical plane; therefore, the joint shear strength should be increased when a significant column axial load is presented. The effect of the column axial load is reflected in the formulas (**Equations 11-1 and 11-2** by NZS 3101-95) for calculating both the horizontal and vertical joint shear reinforcement.

$$A_{jh} = \frac{V_{jh}^*}{\phi_{yh} f_c'} \left(0.5 - \frac{C_j N^*}{A_g f_c'} \right) \quad (2-12)$$

$$A_{jv} = \frac{0.4(h_b / h_c) V_{jh}^* - C_j N^*}{\phi_y f_y} \quad (2-13)$$

$$C_j = \frac{V_{jh}}{V_{jx} + V_{jz}} \quad (2-14)$$

- A_{jh} : Total area of effective horizontal joint shear reinforcement (mm^2)
- A_{jv} : Total area of effective vertical joint shear reinforcement (mm^2)
- V_{jh} : Total nominal horizontal shear force across a joint (N)
- V_{jh}^* : Total horizontal shear force across a joint (N)
- V_{jx} : Total nominal horizontal shear force in X direction (N)
- V_{jz} : Total nominal horizontal shear force in Z direction (N)
- N^* : Minimum designed axial column load at ultimate state (N)
- ϕ : Strength reduction factor
- f_c' : Specified compressive strength of concrete (MPa)
- A_g : Gross area of column section (mm^2)
- h_b : Overall depth of beam (mm)
- h_c : Overall depth of column in the direction of the horizontal shear forces to be considered (mm)
- f_y : Lower characteristic yield strength of non-pressed reinforcement (MPa)
- f_{yh} : Lower characteristic yield strength of horizontal joint shear reinforcement (MPa)

The self-equilibrating truss mechanism, formed by both the horizontal shear reinforcement and column longitudinal bars, is active to carry the induced forces. To sustain this mechanism, a large amount of horizontal and even vertical joint reinforcement is required.

The concrete core supplies the necessary compression field which is balanced by the tension forces in the vertical as well as horizontal reinforcement. The allowable shear stress is limited at $0.25f'_c$ (MPa) for a one-way joint and $0.2f'_c$ (MPa) for a two-way joint.

Again, to control bond deterioration within the joint core, the maximum beam bar diameter passing through the joint core can never be greater than 1/25 of the column depth; similarly the column bar diameter must be less than 1/20 of the beam depth. NZS 3101 [C1] also requires a provision of horizontal confinement to prevent the possible concrete dilation. If a joint is framed at all four column faces, the amount of the transverse reinforcement can be reduced by 50%.

2.3.4.3 AIJ-94 Design Code Requirements

Both the diagonal concrete compression strut and truss mechanisms are adopted by the AIJ standard [A9], and the diagonal concrete compression strut mechanism is taken as the only reliable mechanism throughout a loading history and regardless of bond conditions. However, the reversed cyclic loading will increase the tension strains perpendicular to the diagonal concrete compression strut and finally destroy the concrete strut. At this moment, the concrete strut compression failure cannot be avoided. To prevent such a shear failure, the AIJ standard limits the allowable joint shear stress at $v_u \leq 0.25f'_c$. In addition, extensive joint transverse reinforcement is required to provide confinement to the concrete core. Tension forces in the longitudinal bars can be resisted by an increased concrete compression block, which enhances the diagonal concrete compression strut mechanism. If beams frame into four vertical faces of the joint and if at least two-third of each joint face is covered by framing beams, however, the nominal shear strength of the joint may be enhanced up to 1.3 times $0.33f'_c$.

The truss mechanism might be suspected unless a good bond condition is presented. In general, after several cycles of reversed cyclic loading, bond resistance along the beam as

well as column longitudinal bars starts to deteriorate. Once the bond resistance is almost lost, the truss mechanism is assumed to have diminished. It is recommended that the ratio of the column width to the beam bar diameter must be less than limited values as given by

$$h_c / d_b \geq 0.035 \left(f_y / \sqrt{f_c} \right) \quad (\text{stress unit: MPa})$$

The column axial force effect on the response of the beam-column joints is also addressed by the AIJ standard [A9]. It is believed that the column axial stress smaller than $0.3f_c'$ does not have a beneficial effect on the bond resistance along the beam longitudinal bars within a joint, and that the column axial stress smaller than $0.5f_c'$ does not influence the joint shear strength.

2.4 Previous Studies on Other Essential Aspects Influencing the Response of Beam-Column Joints

2.4.1 Previous Studies on the Seismic Performance of Beam-Column Joints with a Slab

As early as the 1970s, Paulay *et al* [P7] had pointed out that, “When calculating the negative moment capacity, the effect of slab reinforcement immediately adjacent to the beam must be included.” However, in most of early investigations, the test subassemblies did not include a floor slab and so that the laboratorial test data on the behaviour of joints in which a slab was an integral part of the floor beam, were severely limited. Designers often ignored the contribution of floor slabs toward enhancing load resistance, or they might assume an effective slab width as defined in the codes in the case of gravity loads.

Ignoring the slab contribution to the stiffness and strength of beams will most likely result in a significant underestimation of the lateral force resistance. This phenomenon was observed in a full-scaled test on a seven-storey reinforced concrete structure tested at Building Research Institute in Tsukuba. It was found that the measured base shear of the

structure was approximately 70% more than anticipated. The difference between the calculated and measured base shear was due primarily to the contribution of the floor slab bars acting as an additional tension bars to the beam. And also, if the slab effect was ignored, the observed failure mechanism of the structure might be different from that of anticipated.

The mechanism by which the connection develops slab tension has been indicated by Kurose *et al.*, [K2], Cheung *et al.*, [C5], and Pantazopoulou *et al.* [P12], where slab tension is assumed to be equilibrated by loads on the diagonal concrete compression struts. According to this assumption the increased joint shear force is applied directly along the compression zone of the longitudinal beams and is resisted within the joint by an inclined concrete compression strut. The presence of slab enhances the diagonal concrete compression strut mechanism and therefore additional joint shear reinforcement may not be necessary. However, the increased force will induce the failure of the strut. Since slab will produce increased flexural compression forces, thus the flexural deformability of the beam will be reduced plus the enhanced beam flexural strength, which will induce greater shear forces in the beams. Thus increasing of the flexural strength and stiffness of beams may result in enhanced lateral load resistance for the entire structure. This will improve the service level lateral load-response behaviour of the frame and reduce overall deformation and ductility demands. If the stiffness of the structure is underestimated, the structure may also experience unexpectedly high acceleration forces [F1].

In order to better understand the slab effect on the performance of beam-column joints, numerous studies have been carried out recently.

Durrani and White [D1] conducted a test program to study the effect of slab on the performance of beam-column joints. The primary interest was placed on the joint shear stress level and the amount of joint transverse reinforcement. They concluded that at a large deformation level, the amount of joint transverse reinforcement became more important

than the level of shear stresses in the joint. When a structure is subjected to a severe earthquake force, the contribution of the slab in calculating the beam flexural strength should not be ignored.

A series of tests were conducted at the University of Minnesota to investigate the slab contribution to the lateral load resistance [A4]. It can be seen from the results that the factors that affect the width of slab, which acted as a tension flange to the longitudinal beams, were the torsional stiffness of the transverse beams, the shear deformation of the slab, and the rigid-body rotation of the slab. The increase in the torsional stiffness of the transverse elements required the moment-carrying capacity of the specimens to be increased.

Zerbe and Durrrani [Z1] tested four exterior connections to determine the effective slab width and the effect of slab on the confinement to a joint. Based on the experimental works, it was suggested that bars in a slab should be considered effective in calculating the negative flexural capacity of the beam when its width is equal to twice the total beam depth on each side of the beam. The column to beam flexural strength ratio and shear in columns should, accordingly, be based on the increased capacity of the beams. When calculating shear in the joint, the slab bars within the effective slab width should be considered together with the beam top bars. Although the slab bars did not pass through the joint, the compression force in the concrete balancing tension in the top steel was directly applied to the joint.

Extensive experimental investigations including a particular United States-New Zealand-Japan-China cooperative research program were carried out in different countries. In this program, the test specimens contained interior and exterior beam-column joints with slabs. The main variables included loading conditions, the depth of transverse beams and the influence of transverse members, the ratio of the column to beam flexural strength, the amount of joint transverse reinforcement and joint shear stresses.

From these experiments, it can be generally summarized that the tension forces developed across the slab have a significant impact on the flexural capacity of the beam, and also that the presence of slab has been observed to affect the strength, stiffness and shear in the joint. It is well established that one parameter that may affect the amount of the slab bars participating to the lateral stiffness and strength of a reinforced concrete frame is the torsional stiffness of the transverse beam elements. Another parameter that may affect the slab bars participation is the effect of the initial damage to the structure which attributed to the loading in the transverse direction. It is noted that the damage due to lateral loads carried in the transverse direction may have a significant effect on the behaviour of the structure in the longitudinal direction. If the transverse element is damaged from carrying loads in its direction, it may not be as effective in developing the slab participation in the longitudinal direction [F1].

In the case of exterior connections, the stiffness of transverse beams has an important effect on the slab bars participation because it is the only means to resist these tension forces in the slab. Therefore, if transverse beams are strong enough it is apparently capable of developing the full slab tension strength. However, if transverse beams are relatively weaker and fail in torsion, less slab contribution develops. On the other hand, in the case of interior connections, the slab is usually in tension on both sides and therefore, the influence of transverse beams is reduced.

Previous studies indicated that the total effective tension bar area equals to the area of the beam tension bars plus a portion of the slab bars. However, the information regarding the effect of the slab on the beam flexural strength is still limited and therefore it is difficult to estimate an effective amount of the slab bars that may participate in moment resistance due to the following reasons:

- (1) It is depends on the magnitude of earthquake imposed inelastic deformation;
- (2) The contribution of slab bars depends on its anchorage condition;

(3) It also depends on the presence or absence of transverse beams.

Usually, a level of 2% drift presented a reasonable upper limit in the test.

Current codes and recommendations provide guidance for the effective width of slab to be considered as flexural bars for the beam subjected to lateral loadings. These provisions are based largely on the test results of subassemblages comprising columns and beams without floor slabs. The ACI 318 [A2] code recommends that the effective slab width in tension may be assumed to be the smallest

- (1) Beam width plus 16 slab thicknesses;
- (2) The span of the transverse edge beam;
- (3) 1/4 of the span of the longitudinal beam.

The AIJ standard specifies the effective slab width as 1/10 of the beam span. In the NZS 3101 [C1] code, the width of a slab in tension is taken up to as 4 times that of the slab thickness and it also depends on whether an interior or exterior joint with or without transverse beams are being considered.

The effective slab width was investigated by Pantazopoulou *et al* [P11]. In this research, three effective slab widths were investigated including:

- (1) The beam width;
- (2) The ACI effective width;
- (3) $b_w + 3h$, which was proposed by Pantazopoulou *et al*. [P12].

It was report that the ACI 318 [A2] recommended effective width and that proposed by Pantazopoulou *et al*. [P12] are similar for the connections considered in their study,

furthermore, the ACI 318 [A2] recommended effective width is controlled by the last criterion.

2.4.2 Previous Studies on Wide Beam-Column Joints

2.4.2.1 Natures of Wide Beam-Column Joints and Limited Applications

The aforementioned mechanism and observed joint behaviour are performed in conventional frames where the members are designed in a normal range of dimensions. However, in some cases, members are designed with non-typical dimensions to satisfy the architectural configurations. Such examples are: a long-span beam with a considerably larger beam depth than that of the column, shallow beams framed into wall-type columns, and wide shallow beams.

It is well accepted that, under seismic actions, beam moments are likely to be balanced by column moments rather than the opposite beam moments. Therefore it is desirable to place most of flexural bars within the beam web to carry these moments in the joint core. For this reason, in the conventional designed beam-column connections, the beam width is not wider than that of the column. It is recommended [C1] that a beam width b_w , should not be more than the width of the column plus a distance on each side of the column equal to a quarter of the overall depth of the column, and not more than twice the width of the column. This can be written as:

$$b_{w,Maximum} \leq b_c + h_c / 2 \leq 2b_c \quad (2-15)$$

However, this cannot be satisfied within a wide beam frame structure, where the beam width is designed wider than that of the column.

The wide beam frame is taken as a special case somewhere between a conventional beam reinforced concrete frame and a flat plate structure system. The wide beam reinforced

concrete frame buildings and the hybrid structural systems that contain wide beam frames and other structures are traditionally used in non-seismicity regions and are primarily designed as a gravity load resisting frame. The potential application of a wide beam system as a lateral load resisting structure is often ignored considering the current lack of understanding about the seismic performance of this kind of structures. Among some design codes wide beam structures are even restricted to be used in high seismicity regions. In the ACI-ASCE 352 [A3] recommendations no using of wide beam structures was suggested when designing to resist seismic forces. In its provisions, wide beam performance under the earthquake is regarded as a subject needing research. The ACI 318 [A2] design code permits the use of wide beams in seismic regions but restricts the beam width; the restrictions were contained in the ACI 318 [A2] design code, which are actually based on the experience of the code writer rather than on a systematic research. In the NZS 3101 [C1] design code, beams with a large width relative to column width are not permitted to be used in a high seismicity region.

When designing a wide beam frame, geometric restriction is often required based on design practices available up to now. For example, in the USA, the beam width is restricted to be less than $b_c + 1.5h_b$, where h_b is the beam depth and b_c is the column depth, whereas in New Zealand, the beam width is restricted to be less than $b_c + 0.5h_c$ and $2b_c$, where b_c is the column width and h_c is the column depth. However, much research indicated that no beam width limitations are required for the design of wide beam flooring systems in regions of high seismicity if a special detailing regime is adopted [S4].

2.4.2.2 Review of Previous Studies on Wide Beam-Column Joints

The research data on the seismic behaviour of wide beam-column connections were first found in publication in late 1980's. So far, the available data are still not quite abundant.

The main findings from these works on the wide beam behaviour are concisely summarized as follows:

- (1) In 1989, a series of experiments on wide beam specimens were tested by Kajima Institute of Construction. Totally 10 small scaled wide beam-column specimens including six 1/6 scaled interior connections and four 1/2 scaled exterior and interior connections were tested and reported by Hatamoto *et al* [H4]. The primary concerns in this study were the effective beam width and the torsional strength of the transverse beam. It was suggested by Hatamoto *et al.* that the maximum effective beam width should be twice the column width such that all beam bars are expected to yield within 2% of storey deformation. Also, the anchorage ratio of beam bars placed outside the column should be limited in reducing the torsional stress. It was suggested that the torsion strength $\tau_t = 24\sqrt{\sigma_b}$ is a good design criteria for wide beam-column joints.
- (2) Since the early 1990s, a series of experimental investigations have been conducted in the University of Michigan by Wight *et al.* (G2, L6 and Q1). In 1992, Gentry and Wight [G2] reported the experimental results of four 3/4 scaled exterior wide beam connections and also the experimental results were compared to the reinforced concrete theory. In the analytical part, the overall behaviour of the wide beam building and a theory describing a torsional force transfer mechanism were studied. Based on their works, a computer model of wide beam structures was finally suggested. The maximum beam width of $b_w = b_c + 2h_c$ was recommended, which is more liberal than what in the ACI 318 [A2] and NZS 3101 [C1] design codes restrictions and suitable for various connections configuration
- (3) In 1999, LaFave and Wight [L6] presented a number of papers based on the experiments conducted on three 2/3 scaled exterior wide beam-column-slab connections. In their research, the main concern was comparing the behaviour of wide beam system with that of conventional system. LaFave suggested that the limit of

maximum width of a wide beam recommended by ACI 318[A2] should be changed into the smaller one between the two figures calculated from the following equations:

$$\begin{aligned} b_w &= 3b_c, \text{ and} \\ b_w &= b_c + 2h_c \end{aligned} \quad (2-16)$$

This implied that the beam width is related to width and depth of the column. This limit is apparently following the New Zealand approach; however, it is more liberal than the New Zealand code recommendation mentioned as above.

- (4) Three 3/4 scaled interior wide beam-column-slab connections were tested by Quintero-Febres and Wight and reported in 2001[Q1]. In this study, main concerns were the performance of wide beam-column connections which were designed conforming to ACI-ASCE 352 [A3]. The flexibility of wide beam frame building and wide beam system conjunction with structural wall were also investigated in this study. When the beam width was concerned, Quintero-Febres felt that the wide-beam width at interior connections should be limited as follow:

$$b_w \leq b_c + 1.2h_c \quad (2-17)$$

It is noted that this limit, which based on the test observation of interior wide-beam connections, is more restrictive than the limit suggested by Gentry [G2] and LaFave [L6], based on their studies on exterior connections.

- (5) Other researches on the performance of wide beam-column joint included the test on a 1/2 scaled interior connection which was reported by Popov [P5]. Also, two 1/2 scaled interior wide beam-column connections were tested by Stehle *et al.* [S5] at Melbourne and reported in 2001.

Previous investigations on wide beam systems may be divided into two parts: (a) experimental component which investigated the seismic behaviour of individual wide beam-column connection; (b) analytical component and analytical model which were used to study the overall behaviour of wide beam frame building subjected to earthquake loads.

Reviewing the previous research, it can be seen that, in general, the wide beam specimens as well as wide beam frame buildings showed a good performance on lateral load resisting and adequate strength, a good hysteretic behaviour with limited pinching, and also a good energy dissipation capacity and stiffness. Beam hinges deformation, joint shear mechanism were no longer the critical issues leading to failure mechanism. Instead, people put more attention on the portions which located outside of the column faces. These issues include the geometry and confinement of beam outside of the width of the column, the torsion strength of transverse beam, the anchorage of beam longitudinal bars through transverse beam (providing a force transfer mechanism), the spacing and detailing of shear reinforcement in the out portion of wide beam.

Base on these research projects, design recommendations and suggestions for future research were highlighted. In the research carried out by Gentry and Wight [G2], a slight relaxation of the current ACI-ASCE 352 [A3] recommendations on exterior wide beam-column connection was suggested as $b_w \leq b_c + 2h_c$. They also concluded that the wide beam connections performed well even when b_w/b_c was greater than 3. From the current review of the previous investigations, it was found that there were three test results performed by Hatamoto *et al.* [H4] and Stehle *et al.* [S5] exceeding the limitation.

Insufficient bar anchorage or bar slip will lead to pinching of the hysteretic loops. The recommendation from ACI-ASCE 352 [A3] for the anchorage of column bars in a connection is:

$$\frac{h(\text{beam})}{d_c(\text{column bars})} \geq 20 \quad (2-18)$$

A similar ratio is also used to limit the slip of beam bars through columns. In general, the bond condition of the beam bar, which is in term of the ratio of the beam bar diameter over column depth, is easier to be reached than column bar bond condition. This is because that

the wide beam depth is shallower than the normal beam. Gentry and Wight [G2] suggested relaxing this ratio to 16 based on their research but at the same time, pointing out that further research is needed to clarify the role of column bar slip in the performance of beam to column connections.

Other impacts on the performance of wide beam system are attributed to column moment to beam moment ratio, low beam stiffness as well as overall stiffness due to a shallow beam depth and bond loss of the column longitudinal bars. Seismic design provisions specify a principle of “strong column-weak beam” to make sure that beam hinges can be formed at an inelastic stage to dissipate the energy and prevent the development of column hinges. In general, the philosophy of “strong column-weak beam” can be met in wide beam buildings due to the lower moment inertia of wide beams. However, the non-typical characteristic of wide beams will lead to a lag in yielding among the distributed longitudinal reinforcement in the hinge regions.

Base on the review of the previous studies on wide beam-column joints, the potential problems which worth to be investigated as follows:

- (1) Limitation of beam width to column width ratio and moment ratio
- (2) Percentage of beam bars anchored in the column
- (3) Permissible level of joint shear and deformation
- (4) Reinforcement detailing
- (5) Bi-directional loading
- (6) Column aspect ratio, traverses beam width.
- (7) Column axial load effect

2.5 Summary

In this chapter, the current available design and research experiences on characteristics and deficiencies of the traditional gravity-load-designed reinforced concrete frames when they were subjected to earthquake conditions are primarily summarized. The main conclusions from this chapter could be concisely described as follows:

- (1) The reinforced concrete buildings designed for gravity loads only have some inherent capacities to resist earthquake actions. However, the inadequate ductility consideration and the failure mode assumption of the system must be further investigated and get improved. Some researches on the gravity-load designed or lightly reinforced beam-column joints are reviewed and it is found that the joint transverse reinforcement plays a significant role in the performance of such kind of beam-column joints. The bond condition and column axial forces also have influence on the joint shear resisting.
- (2) The joint shear mechanism is reviewed in this chapter followed by a definition of the nominal joint shear stress demands. Design recommendations on beam-column joints are introduced and also the design limitations by different design codes are summarized in details.
- (3) Finally in this chapter, the slab effect on the performance of beam-column joints and previous studies on the wide beam-column joints are reviewed.

Chapter 3

Analytical Studies on Reinforced Concrete Moment Resisting Frames

3.1 Introduction

Features and deficiencies of gravity-load-designed reinforced concrete frame structures have been outlined in Chapter 2. As a consequence of an inadequate lateral load-carrying capacity and poor reinforcing details compared to the modern seismic design code requirements, the damage and seismic hazards confronting these buildings were reported during recent earthquakes such as the 1995 Hanshin-Awaji [C2] and the 1999 Kocaeli [H2],[K1] earthquakes.

Concerning the seismic performance of existing buildings, a lot of attentions have been paid to this subject in recent years with a primary interest upon the evaluation of the performance of non-ductile structures or structures with limited ductility, particularly when they are undergoing an earthquake attack. In general, assessment is applied by means of evaluating the structure's lateral load-carrying capacity, calculating the global or inter-storey drift, tracing the potential damage and identifying the weakpoints of the structure or a failure mechanism. Recently, seismic assessment techniques and guidelines have been recommended by a number of technical documents, such as the Japanese Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings. Eurocode 8 [E1], [E2] is also at the beginning to address the strengthening and rehabilitation of the existing structures.

In this chapter, the seismic performance of lightly reinforced concrete frame buildings will be evaluated by using a finite element analysis approach. The analytical models are set up on basis of the prototype experiments performed by Calvi *et al.* [C6] and Vecchio *et al.* [V1] respectively. The frames are pushed following the loading sequences, which were used during the test corresponding to the predetermined probability of various degrees of damage. By studying these results and comparing them with experimental findings, the seismic behaviour of non-ductile or limited ductile structures is presented in details

3.2 Summary of the General Seismic Assessment Methods and Procedure for Existing Reinforced Concrete Structures

The current design practice is incorporated with the performance-based design criteria. In order to pre-estimate the performance of a newly designed structure subjected to a specified ground motion, a systematic approach is recommended in which the flexural response could be obtained by employment of appropriate capacity design principles. However, the global response of the existing structures must be determined with consideration of their relative strength of the sections, joints, members and stories, which cannot be assumed at the beginning of the process. Therefore, it is an essential challenge to the researchers to assess the seismic performance of existing buildings.

Varied methods ranging from a checklist approach to a realistic assessment procedure are available to assess the seismic capacity of structures. The assessment can be achieved either by means of visual inspections or by complex analyses with the assistance with powerful and robust computer programs. Each approach can provide specific information on different degrees of accuracy. Among of these, non-linear time history analysis is a viable method for assessing the response of reinforced concrete structures especially for those special, complex and important structures. Nevertheless, as a consequence of the numerous uncertainties involved and time-consuming, non-linear time history analysis is considered

to be appropriate for research purposes rather than using it in every day design and general seismic assessments. Meanwhile, non-linear static analysis is commonly used to evaluate the seismic capacity of structures.

When assessing existing buildings, the evaluation of responses rather than the strength of the structures are usually more importance. In a displacement-based assessing procedure, the structure is pushed towards a set of specified limits states including a collapsed state. At each state, the global as well as local performance is detected. Therefore a relationship could be established between the performance levels and the damage to the structure. Furthermore, the performance-based assessment emphasizes the evaluation of the structural capacity rather than the seismic demand. The performance of the existing structures can be illustrated by a capacity curve in terms of the loads versus displacement relationship. Some of performance levels are defined by SEAOC, 1995 [S1] as follows:

- (1) Collapse prevention which allows a small margin of safety against collapse during a severe earthquake.
- (2) Life safety which allows significant damage to the building's lateral force resisting system, but keeping a substantial margin against collapse.
- (3) Immediate occupancy where relatively minor damage occurs to the building. The lateral force resisting elements should retain their initial strength and much of their original ductility.

Apparently, weaknesses of structural system and deficiencies of reinforcing details in the existing RC buildings have a strong influence upon the potential damage and failure modes of the buildings during an earthquake. In a performance-based assessment process, it is an essential step to evaluate the strength and deformation capacity of the critical sections and members of the buildings. The flexural strength and shear strength of members must be carefully checked to determine whether the flexural or shear failure will first occur. In addition, the investigation of the strength of joints as well as the relationship between the

strength of the beams and columns framing into the joint would be needed. A fundamental aspect of these steps would be the identification of the inelastic deformation mechanism and whether a beam-sway or column-sway mechanism is more likely to be formed.

3.3 Description of Analytical Frame Modelling

Two-dimensional finite element models of the test frames would be used and developed in this study in describing the seismic behaviour of lightly reinforced concrete frames. In the following sections, design and test methods of the test frames are first summarized and then the finite element modeling would be introduced.

3.3.1 Description of the Designed Frame by Calvi *et al.*

An experimental test on a three-storey reinforced concrete frame building, which was subjected to a quasi-static cyclic loading, was conducted by Calvi *et al.* [C6]. The prototype building was a moment-resisting frame system consisting of three storeys and three bays, which was designed for resisting gravity loads only. The design principle and reinforcing details presented a typical construction practice in Italy between the 1950's and the 1970's.

The geometries and reinforcing characteristics of the test frame were 2/3 scaled to the prototype. Therefore, the total height of the test frame was 6m with an inter-storey height of 2 m and a center-to-center span of the bays being 3.01 m, 1.34 m and 2.33 m respectively. The geometries and reinforcing details of the test frame are shown in **Figure 3-1**. In this test frame, design and reinforcing techniques were following a gravity load design philosophy, and no transverse reinforcement was placed in the joint cores. Smooth round bars were adopted for both the longitudinal and transverse reinforcement in the beams and columns. The end of beam longitudinal bars in the exterior joints was not bent. The lap splices were placed in the beam bars within the interior joints and in the column bars just above floor level.

The test frame was designed with concrete compression strength of 16 MPa. The concrete strength of the first storey columns was actually higher than other frame members. The reason was that after the casting of the first storey, the concrete characteristic was modified to increase the workability for the next casting. Test results of concrete and steel specimens can be found in the published paper by Calvi *et al.* [C6], and they are summarized in **Table 3-1**.

Table 3-1 Material properties used in the frame tested by Calvi *et al* [C6]

Material		f'_c (MPa)	f_y (MPa)	f_u (MPa)
Concrete		14.1	—	—
Steel	$\phi 8$	—	385.6	451.2
	$\phi 12$	—	345.9	458.6

A quasi-static loading history was applied on the frame in terms of a horizontal displacement process provided by a screw-actuator on the top of each story, respectively (see **Figure 3-2**). The loading history consisted of a series of cycles with a top drift ratio ranging from 0.2% to 1.6% in both the positive and negative directions (see **Figure 3-3(a)**). Three cycles were applied at all the drift ratios except one cycle applied at the drift ratio of 1.6% in each test. During the test, the top jack was a “master” one to control the displacement and the other two were “slave” ones, which were used to maintain the desired load patterns. An inverted triangular loading distribution was utilized which was in accordance with the design code, and the proportion was kept constant during the test (see **Figure 3-3(b)**). This lateral loads distribution was assumed to be a linear form of the first vibrational mode shape. In addition, gravity loads were applied resulting in a total weight of 73 kN for the first and second floors and 54.2 kN for the top floor. **Figure 3-2** shows the test setup of the experiment.

3.3.2 Description of the Designed Frame by Vecchio *et al.*

In 1992, Vecchio and Emara [V1] carried out an experiment to study the behaviour of a reinforced concrete frame with one bay and two storeys. Of particular interest in this test was the magnitude and influence of the shear deformation to the frame.

The test frame was designed with a center-to-center span of 3500 mm, a storey height of 2000 mm, and the total height of 4600 mm. All frame members were 300 mm wide by 400 mm deep with uniform reinforcing details as shown in **Figure 3-4**. It was not mentioned in their paper for what kind of a design philosophy was followed in this particular frame design. Due to the uniform cross-sections and reinforcing details for all members, the column to beam flexural strength ratio was taken as 1.0. If strain hardening of the beam bars was taken into account, a “weak column-strong beam” conformation could be expected. The reinforcing details and material properties used in this test are tabulated in **Table 3-2**.

The test frame was subjected to the combined column axial loads and lateral loads. The column axial load with a magnitude of 700 kN was applied on each column and the lateral load was applied with a monotonically increased manner. The test was completed within several days, At the end of each day, the test frame was unloaded, and on the following day, the initial load level was approximately the same as the final load level achieved in the previous day. This loading procedure resulted in a particular loading sequence including unload and reload paths. **Figure 3-5** illustrates the test setup of the experiment.

Table 3-2 Reinforcing details and material properties used in the frame tested by Vecchio *et al* [V1]

Material	Reinforcing details		Material properties	
Beam bars	Top	Bottom.	f_y (MPa)	f_u (MPa)
	4 No.20	4 No.20	418	596
Column bars	8 No.20			
Transverse reinforcement	No.10@125		454	640
Concrete	—		30 (MPa)

Figure 3-1 Geometries and reinforcing details of the test frame (after Calvi *et al.*)

Figure 3-2 Test setup (after Calvi *et al.*)

Figure 3-3 Loading sequences and loading patterns (after Calvi *et al.*)

Figure 3-4 Geometries and reinforcing details of the test frame (after Vecchio *et al.*)

Figure 3-5 Test setup (after Vecchio *et al.*)

3.3.3 Finite Element Modelling

Before FEM techniques were currently well developed, structural performance was observed mainly through experimental approaches. Since Ngo and Scordelis [N1] applied finite element technique in analyzing reinforced concrete structures, much research interests have been concerned with FEM analysis of concrete structures. Associated with the advantages of computing technology, the computer models and numerical simulations become much more suitable for assessing the seismic performance of reinforced concrete structures, particularly for the use of non-linear finite element analysis which may give a better understanding of the mechanisms within a structure during loading to failure.

Up to now, the finite element analysis approach is widely used in seismic assessment procedures. However, most applications are based on relatively simplified models, where frame elements are used for the analysis of concrete structures under cyclic loadings. In these simplified models, columns and beams are modeled with beam elements and beam-column joints and hinges are modeled with spring elements to take into account of shear deformation. Therefore, via an element-based approach, the seismic performance of reinforced concrete structures can be investigated on a macro-level analysis. In order to assess seismic performance such as the deterioration of the shear-carrying capacity and limited flexural ductility, the fracture behaviour of reinforced concrete structures under cyclic loadings must be estimated in detail. It would be difficult of course to reach the goal with a frame-based approach.

Alternatively, a plane stress element model can be used to describe the overall behaviour and failure mechanisms, as well as the individual members' deformation components of flexure and shear. The model can take into account of the compatibility, equilibrium, and the constitutive properties of the different materials including a reasonably accurate prediction of the fracture behaviour of reinforced concrete structures.

A Finite Element Model of the Frame

In current analyses, the test frames were analyzed using the finite element program DIANA 7.2 [D3]. A two dimensional concrete model was developed using an 8-nodes quadrilateral plane stress element with a 2×2 Gauss integration scheme. The reinforcement was presented by a 2-nodes truss element. (See **Figure 3-6**)

Figure 3-6 Truss element and plane stress element

The finite element modeling was conducted using a total strain model associated with the fracture energy expression based on the parabolic compression hardening-softening relationship for the concrete. The concrete tension strength f_t used in the analyses was determined from the concrete compression strength f_c according to CEB-FIP Model Code [C2]. The cracking of the concrete was developed using the smeared crack concept with rotated cracks and the reinforcement was modelled with Von Misses yield criterion augmented with elastic-hardening fracture behaviour in tension and compression. Detailed descriptions of constitutive model can be found in the DIANA manuals and will be discussed in later sections.

The finite element models were discretized to match the geometries of the test frames. The gross dimensions and material properties were chosen to coincide with those of the as-built subassemblies. The elements' discretization and reinforcement layout for the two analytical models are shown in **Figures 3-7** and **3-8**, respectively. It should be noted that the meshes were refined near the joints in Calvi's test structure.

The analytical frames were subjected to lateral loadings in terms of a series of increased displacements in a monotonic manner. The locations and magnitude of the lateral loadings were carefully chosen to match the applied loads during the tests.

Constitutive Models

To develop a proper model, a tensile and compressive yield surface associated with hardening and softening and cracking behaviour should be investigated. Usually mechanical properties used in finite element analysis are usually obtained from experimental investigations as well as being recommended in the FEM code.

Modelling plasticity of materials involves the use of a yielding criterion, a flow rule, and a hardening rule. Either the Von Mises or Drucker-Prager yielding criterion could be used for modelling the concrete and reinforcement in compression. In tension, the concrete is assumed as a brittle or a softening material with smeared cracks.

Descriptions of un-loading and re-loading relationship for the materials should also be specified in the constitutive. The constitutive laws of the concrete, steel and bond behaviour are developed based on the plasticity theory taking into account of the permanent strain as well as the unilateral behaviour of the cracked concrete. The constitutive laws will be described in detail in the following chapters but not in this chapter, because the structures here were loaded in a monotonic manner.

Figure 3-7 Finite element model and mesh scheme for Calvi's frame

Figure 3-8 Finite element model and mesh scheme for Vecchio's frame

Modelling of Concrete in Compression

The response of the concrete in compression is usually taken into account through an elastic-plastic model. In a finite element approach, the elastic-plastic stress state can be limited by different yielding criteria such as an elastic response, Von Mises yielding criterion, Drucker-Prager yielding criterion, or a total strain formulation. Once yielding occurs, an associated flow rule with isotropic hardening will be used. By applying different criteria to evaluate the response of structures, it is found that the concrete compressive capacity and the overall response of the structure can be more accurately estimated using the total strain criterion. Therefore, this criterion was adopted for the concrete compressive response in the all investigations conducted in this and subsequent chapters.

In the DIANA 7.2 manuals, seven pre-determined models are available to present the concrete responses in compression. Some models are shown in **Figure 3-9**. Others can be found in the DIANA manuals including an elastic model and a multi-linear model. Users also can introduce their own models via user defined sub-routines.

Figure 3-9 Modelling of concrete response in compression

Modelling of Concrete in Tension

The tension yield surface of the concrete typically consists of either a constant cut-off (Rankine yield surface) determined from the concrete tensile capacity or a bilinear envelope, which was determined from the uniaxial tensile strength and the uniaxial compressive strength in the tension-compression regime, or combined with the Rankine yield surface in the tension-tension regime (see **Figure 3-10**). The bilinear yield surface was used in current investigations.

Tension softening behaviour consisting of various models, including the elastic model, the ideal model, the brittle softening model, the linear softening model, the exponential softening model, the Hordijk [H3] model and the multi-linear model, are pre-defined in the DIANA [D3] manuals. Some of the models are shown in **Figure 3-11**. The tensile stress-strain curve in these models consists of two segments: a hardening branch and a softening branch. The hardening branch is made up of a linear segment with a slope equal to the concrete elastic modulus. It is shown from previous research that the un-reinforced concrete subjected to tension was typically failed with a non-linear decaying branch. This decay was controlled by the fracture energy density of the concrete, G_f (fracture energy is calculated by CEB-FIP Model Code). The concrete tensile strength f_t used in these analyses was determined from the concrete compressive strength f_c according to CEB-FIP Model Code [C2].

By evaluating different tension softening models and comparing the analytical results with the experimental ones, Natio *et al.* [N3] indicated that Hordijk's tension-softening model provided a good approximation of the measured un-reinforced concrete tension-softening behaviour. Therefore, the Hordijk's model was selected as a proper model for the softening branch of the tension.

Figure 3-10 Tension cut-off models

Figure 3-11 Modelling of concrete response in tension

Modeling of Concrete Cracks

Once the concrete tensile capacity is reached, concrete cracks and loss of the concrete tensile strength cannot be avoided. In the finite element simulation this behaviour must be properly addressed. Usually in a plane stress finite element, the smeared crack model has been used extensively for modelling cracking and so for practical engineering purposes, this becomes a general approach in finite element formulations.

Two methods based on the smeared crack concept including the fixed-crack model and the rotating-crack models are given in the DIANA [D3] manuals. According to the recommendations in the DIANA [D3] manuals, the adopted rotating-crack model is based on the concept of the total strain. It is assumed that the cracks are always oriented normally to the diagonal tensile strains when the cracking stress in the concrete was exceeded. Since the rotating-crack model can provide a good estimation of response as well as a more rational formulation, it was adopted for the all investigations in this thesis.

Modelling of Reinforcement

According to the recommendations given in the DIANA [D3] manuals, reinforcement can be modelled as embedded steel elements or separate steel elements in the form of truss elements. The Von Mises yield criterion with associated flow and isotropic strain hardening models are used to describe the constitutive behaviour of the reinforcement. In the current analyses, the truss elements were used in the entire system.

The steel is taken as a strain hardening material with the hardening rule based on the uniaxial stress-strain response from material tests. The stress-strain curve is controlled by four parameters: Young's modules; hardening modulus; yield stress and strength. In addition, the tension and compression responses are assumed being identical.

The truss model allows for the investigation of bond-slip behaviour of the reinforcement with respect to the surrounding concrete. In this case, the truss elements must be attached to the adjacent concrete elements through a series of interface elements. However, introducing of interface elements leads to the requirement of a double nodes solution at the location of the reinforcement. As a result, considering of the bond-slip behaviour in the frames makes the analyses more computationally expensive. In current applications, the interaction between the steel and concrete was assumed to occur via perfect bond.

Solution Strategy

The commonly used solution strategies for the finite element investigation include regular Newton, modified Newton, and a linear scheme, among these, the regular Newton iteration has been proved to be the most efficient one when using the total strain formulation and is suitable for simulating the moderate non-linear behaviour where the minimal damage (cracking or plasticity) in each increment is expected. For this reason, the regular Newton iteration was used in accepted in the current analyses.

3.4 Comparison of Calvi's Experimental and Analytical Results

3.4.1 Validation of Test Results

Deformation

Figures 3-12 and 3-13 illustrate the frame deformation under difference loading manners. During the test, the distribution of lateral loads was determined based on the designed code according to a linear first vibrational mode shape. These lateral loads were applied in a hybrid-displacement controlled manner, where the top screw jack actuator acting as a “master” to directly control the top floor displacement associated with the other two “slave” actuators to maintain the required lateral load distributions [C6] (refer to **Figure 3-3**). Since the floor masses were not equal each other, the distribution of the lateral loads was normalized to the top actuator force, which results in:

$$\{\bar{F}_i\} = \begin{Bmatrix} F_1 \\ F_2 \\ F_3 \end{Bmatrix} = \begin{Bmatrix} m_1 a_1 \\ m_2 a_2 \\ m_3 a_3 \end{Bmatrix} \quad (3-1)$$

$$\frac{\{\bar{F}_i\}}{F_{i1}} = \begin{Bmatrix} 1 \\ F_{i2}/F_{i1} \\ F_{i3}/F_{i1} \end{Bmatrix} = k \begin{Bmatrix} 1 \\ (m_2/m_1)h_2/h_1 \\ (m_3/m_1)h_3/h_1 \end{Bmatrix} \cong k \begin{Bmatrix} 1 \\ 0.88 \\ 0.46 \end{Bmatrix}$$

Figure 3-12 Deformed shape of Calvi's frame under gravity loadings

Figure 3-13 Deformed shape of Calvi's frame under combined column axial and lateral loadings

After several plastic loading cycles at a large deformation level, strength degradation could be expected resulting in decreased segments in the loading history. Loading reduction would be taken as an un-loading in the analytical program, and therefore in finite element analysis, a loading controlled method could not be applied directly especially in the plastic range. Instead, a displacement controlled loading pattern was adopted in the analyses.

In the experiment and finite element analyses, gravity loads were applied in terms of the distributed node loads in the beams, which simulated the imposed live loads.

Load versus Displacement Relationship

The load versus horizontal displacement relationship under monotonic loadings is shown in **Figure 3-14**. Before the frame was loaded into a drift ratio of 1.2%, both the analytical and experiential results were much closer in either loading direction. When the frame was pushed into the positive displacement beyond a drift ratio of 1.2%, the analytical frame basically retained its strength with only a slight reduction, whereas the strength degradation in the test frame was observed apparently at this stage. The difference between the experimental and analytical results might be attributed to the material properties and modeling approaches adopted in analysis, where the cyclic behaviour was not taken into account when modeling the structure. During the test, pinching behaviour was observed resulting in the strength and stiffness degradation after loading runs. Pinching behaviour was mainly due to the slippage of the beam and the column smooth round bars and as a consequence of strain penetration and bond deterioration in the joint region [C6]. However, the current finite element approach cannot simulate the pinching behaviour for the above mentioned reasons, the information regarding energy dissipation and equivalent viscous damping therefore cannot be provided.

Figure 3-14 Comparison of hysteretic behaviour between experimental and analytical results under monotonic loading history

An analysis with a reversed cyclic loading history was also conducted with attempting to refine the analytical results. The load versus horizontal displacement relationship under reversed cyclic loadings is shown in **Figure 3-15**. Up to the second loading cycle, when the loading on the frame reached a drift ratio of 0.6% in either direction, the analytical results coincided with that obtained from the test results. But after the loading on the frame running into the negative displacement equaled to a drift ratio of 1.2%, the strength and stiffness degradation became obvious. The calculated capacity at this stage was approximately only 80% of the measured result. In the following semi-cycle, where the frame was pushed to a drift ratio of 1.6%, the calculated capacity matched the test value and demonstrated the strength degradation. At this stage, the significant pinching behaviour still could not be found from the analytical hysteretic loops.

Since the monotonic curve can be taken as the skeleton of hysteretic loops, in the following sections, the discussion is based on the analytical results under the monotonic loadings.

Figure 3-15 Comparison of hysteretic behaviour between experimental and analytical results under reversed cyclic loading history

Damage and Plastic Hinges Development

During the test, extensive damage was found in the joint regions and beam-column interfaces. A “concrete wedge” mechanism in the exterior joints and the losing of the load-carrying capacity on the beam bar end-hooks were observed. However, these phenomena could not be simulated by the finite element approach because the bond behaviour was ignored in the current analytical models and the beam longitudinal bars were assumed as continuous reinforcement passing through the joint core. In addition, the beam bar end-hooks in the exterior joints also could not be properly modeled. However, the cracking development could be traced via the finite element approach.

It was reported by Calvi *et al* [C7] that the damage mostly generates in the joint regions (exterior tee-joints) or at the beam-column interfaces with a wide flexural crack. **Figure 3-16** shows the cracking patterns via finite element analysis. Cracks were concentrated in the joint regions and column ends on the first and second floors. On the first floor, both the

interior and exterior joint panels had extensive cracks, whereas on the second floor only the moderate cracks could be found in the joint cores.

Figure 3-16 Cracking patterns of Calvi's frame via finite element analysis

According to Calvi *et al.* [C7], the global mechanism of the damage was the joint damage combined with the column hinging at the bottom end sections of the columns on the first floor. As a result, a “shear hinge” and a hybrid mechanism were proposed by Calvi *et al.* [C7]. The pure soft storey mechanism, if not be prevented, could be postponed due to the presence of the “shear hinge.” These assessments were confirmed in the current analysis.

In the analysis, strains and yielding in the longitudinal bars were checked. The hinge patterns at different drift ratios are shown in **Figure 3-17(a)** with the corresponding ductility demands being shown in **Figure 3-17(b)**. At a drift ratio of 0.2%, the tension yield occurred in the columns on the first floor. From a drift ratio of 0.25% to 0.4%, the extensive tension yield was observed in both the column ends on the first floor as well as on the second floor. It was noticed that the intermediate column bars started to yield in tension at this stage as well. When the frame was pushed into a drift ratio of 0.85%, the first compression yield was found in the intermediate column base and consequently, the

compression yield developed with the increase of the loading level. Up to a drift ratio of 1.3%, column hinges were fully developed at the four column bases as well as at the top ends of the intermediate columns on the first floor. During the analysis, however, no beam bars yielding was found and the plastic hinges on the face of the column never developed.

Figure 3-17 Hinge patterns and ductility demands in Calvi's frame via finite element analysis

3.4.2 Discussion of Analytical Results

3.4.2.1 Overall Behaviour

From the tested hysteric loops and the analyzed capacity curve, the yield displacement occurred approximately at an equivalent drift ratio of 0.4%. By checking the yield status of the reinforcement, it was found that stresses in the beam bars did not exceed the yield level at this stage. Column bar yielding in tension did not result in the soft storey mechanism. Non-linear behaviour was mainly attributed to the joint shear deformation.

Due to a large reduction in the overall capacity, the test was interrupted after the first single cycle at a 1.6% drift ratio. At this moment, the measured load-carrying capacity was reduced to 80% of the maximum capacity. In finite element analysis, the load on the frame finally reached a drift ratio of 2.0%, after that the analysis became instable and auto-terminated at this stage. The obtained load-carrying capacity by the finite element

approach represented different outcomes depending on whether a monotonic or a reversed cyclic loading history was applied. In general, the loss in load-carrying capacity is mainly due to the extreme joint panel damage and $P-\Delta$ effect.

3.4.2.2 Joint Shear Behaviour

Typical bending moment diagrams of the beam which are separated from the frame undertaking different loads are illustrated in **Figure 3-18 (a)** to **(c)**. The large magnitudes of bending moments at the beam ends close to column A and column C will induce significant shear forces in the joint cores. In the test, the exterior tee-joint and interior joint on the first floor suffered severe damage. Therefore, the exterior joint panels in column A and the interior joint panels in column C were taken as the critical regions in this frame and were evaluated in the current study. In total, six joint panels were investigated via finite element analysis and among these four types of joint configuration were included which were an exterior tee-joint, a corner joint, an interior joint and an interior tee-joint (see **Figure 3-19**). The exterior joints were assigned as Joint A1 to Joint A3. Similarly, the interior joints were assigned as Joint C1 to Joint C3. The number indicated the floor at which the joint was located.

Joint Shear Stress

The code recommendation on the joint shear stress calculation is based on the equilibrium of the isolated beam-column connection with the assumption that the longitudinal bars at the same level are strained with identical magnitude. In the finite element approach, joint shear stresses are calculated directly from the stress output on the element nodes or Gaussian points. **Figures 3-20** and **3-21** show the nominal shear stresses in the exterior and interior joint panels via finite element analysis, respectively. The recommended joint shear strength limited by ACI-ASCE 352 [A5] is also shown in these two figures.

Figure 3-18 Bending moment diagram of Calvi's frame under different loadings

Figure 3-19 Different joint types

The maximum shear stress was found in either the exterior or interior joint panel on the first floor. For each individual joint panel, the maximum joint shear stress was obtained approximately at a 0.2% drift ratio, and after that the shear stress level dropped significantly indicating the depletion in the joint shear-carrying capacity. In the subsequent loading cycles, the shear stress level remained at a limited magnitude of variation. It can be seen that the shear stresses in both the exterior and interior joints were very much lower than that from the ACI-ASCE 352 [A5] recommendation, which is $1.25\sqrt{f'_c}$ (f'_c unit in MPa).

Figure 3-20 Nominal shear stresses in the exterior joints of Calvi's frame via finite element analysis

Figure 3-21 Nominal shear stresses in the interior joints of Calvi's frame via finite element analysis

The maximum shear stress in Joint-A1 (an exterior tee-joint) was 2.18 MPa, which equals $0.6\sqrt{f'_c}$. On the second floor, the maximum shear stress in Joint-A2 (an exterior tee-joint) was 1.33 MPa or $0.36\sqrt{f'_c}$. In Joint-A3 (a corner joint), the maximum shear stress was only 0.4 MPa or $0.1\sqrt{f'_c}$. The maximum shear stress in the interior joints, from Joints C1 to C3, was 1.9 MPa, 1.1 MPa and 0.25 MPa or 0.5, 0.3 and 0.1 in terms of $\sqrt{f'_c}$, respectively. Comparing the results of the exterior joints with those of the interior joints, it was found that the shear stresses in the exterior joints were higher than those in the interior joints at the same loading levels. Therefore, the exterior joints were expected to undertake a large magnitude of shear force and resulted in a more severe damage than what would be in the interior joints.

Diagonal Joint Compression Stress

Diagonal joint compression stress can be calculated by:

$$f_{dc} = \frac{f_c}{2} + \sqrt{\left(\frac{f_c}{2}\right)^2 + v_{jh}^2} \leq 0.5f'_c \quad (3-2)$$

where

f_{dc} : Diagonal joint compression stress (MPa)

f_c : Concrete compressive stress. (MPa)

v_{jh} : Nominal horizontal joint shear stress (MPa)

f'_c : Concrete compressive strength (MPa)

Figure 3-22 Diagonal compression stresses in the exterior joints of Calvi's frame via finite element analysis

Figure 3-23 Diagonal compression stresses in the interior joints of Calvi's frame via finite element analysis

Figures 3-22 and 3-23 show the diagonal compression stresses in the exterior and interior joints, respectively. Also shown in the Figures are the curves of principal diagonal compression strength limited by CEB [C4] category 2, which is $0.5f'_c$ MPa. In general, diagonal compression stresses on the first and second floors in either the exterior or interior joint panel were very much higher than those on the third floor.

With a drift ratio of 0.2% as in the earlier situation, diagonal compression stresses in Joint A1 exceeded the diagonal compression strength with the obtained diagonal compression stress being of $0.56f'_c$. The maximum diagonal compression stress in Joint A1 was found at a drift ratio of 1.3%, which was $1.05f'_c$ MPa and after that, the diagonal compression stresses in Joint A1 decreased quickly. On the second floor, diagonal compression stresses in Joint A2 gently increased from a top drift ratio of 0.2% to 1.3% with the diagonal compression strength being reached at a drift ratio of 0.6%. Beyond the drift ratio of 1.3%, the diagonal compression stress curve became much steeper. The maximum stress was obtained at the end of the analysis, which was $1.2f'_c$. Joint A3 showed a desired behaviour, where the diagonal compression stresses remained at a lower level until the end of the analysis with the maximum value being about $0.15f'_c$.

Diagonal compression stresses in the interior joints presented a similar response to those in the exterior joints. The maximum diagonal compression stress in Joint C1 was $1.15f'_c$ MPa, which was slightly higher than that in Joint A1. On the second floor, the maximum diagonal compression stress in Joint C2 was $1.13f'_c$. Joint C3 demonstrated a desired behaviour, where the diagonal compression stresses also remained at a lower level until the completion of the analysis and the maximum stress was about $0.16f'_c$, which occurred at the beginning of the analysis.

At early stages, diagonal compression stresses in the joint panels on the first floor were higher than the corresponding joint panels on the second floor. This might be attributed to the large magnitude of the induced nominal joint shear stress, which was carried by the concrete compressive strut. Once Joints A1 and C1 lost their diagonal compression strength, the compression stresses in Joints A2 and C2 increased dramatically. This showed that the diagonal compressive failure leading to loss of joint shear strength in some joints therefore the inter forces and deformation demands redistributed in the frame members.

Diagonal Joint Tension Stress

Based on the studies by Priestly and Calvi [P2], the diagonal tension strength of concrete may conservatively be taken as $0.29\sqrt{f'_c}$. Thus the diagonal joint tension stress can be limited by:

$$f_{dt} = \frac{f_c}{2} - \sqrt{\left(\frac{f_c}{2}\right)^2 + v_{jh}^2} \leq 0.29\sqrt{f'_c} \quad (3-3)$$

where

f_{dt} : Diagonal tension stress (MPa)

f_c : Concrete compression stress (MPa)

v_{jh} : Nominal horizontal joint shear stress (MPa)

f'_c : Concrete compressive strength (MPa)

Figure 3-24 Diagonal tension stresses in the exterior joints of Calvi's frame via finite element analysis

Figure 3-25 Diagonal tension stresses in the interior joints of Calvi's frame via finite element analysis

Figures 3-24 and **3-25** show the diagonal tension stresses in the exterior and interior joints, respectively. In general, diagonal tension stresses remained at a low level during the analysis. The maximum diagonal joint tension stresses at the drift ratio of 0.2% were $0.16\sqrt{f'_c}$, $0.08\sqrt{f'_c}$ for Joints A1, A2 and $0.13\sqrt{f'_c}$, $0.05\sqrt{f'_c}$ for Joints C1, C2, respectively. Diagonal tension stresses in the corner and interior tee-joints remained at a much lower level during the whole analysis.

3.4.2.3 Joint Deformation

Significant joint damage will result in a large joint deformation demand. In this study, joint shear deformation was evaluated by investigating joint shear distortion. The average joint shear distortion was calculated by **Equation 4-5** (refer to **Figure 4-17**), which was recommended by Hakuto [H1].

In this study, the total displacement, $(\Delta X_i^j, \Delta Y_i^j)$ at each node of the joint panel was first picked up from the analysis results. Here, “*i*” represents the number of the joint panel node, which ranges from 1 to 4; “*j*” stands for the number of loading step. Adding the nodal displacement to its original coordinate values, the updated nodal coordinate values can be obtained. The diagonal length of a joint panel must be calculated from the updated nodal coordinate value. By applying **Equation 4-5**, joint shear distortion can be calculated. It should be noted that only the joint panels on the first floor were evaluated because the prior analysis showed that these joints suffered extensive damage.

Figure 3-26 shows the joint shear distortion demands versus horizontal displacement in different top drift ratios in reference to the indicative values of γ suggested by Pampanin *et al* [P4] as follows:

- (1) Undamaged (uncracked): $\gamma < 0.002$
- (2) Limited damage: $0.002 \leq \gamma < 0.005$

- (3) Extensive damage: $0.005 \leq \gamma < 0.01$
- (4) Critical damage (reparability issues arise): $0.01 \leq \gamma < 0.015$
- (5) Incipient collapse: $\gamma > 0.015$

Figure 3-26 Joint shear distortion of Calvi's frame

It was reported by Calvi *et al.* [C6] that the exterior joints were subjected to higher rotation demands than to the interior ones, which may reach to $\gamma = 0.006$ - 0.008 corresponding to an extensive but not critical level of damage. The analytical results presented different joint shear deformation responses. From **Figure 3-26**, it can be seen that both the interior and exterior joints were in between limited damage and extensive damage at a drift ratio from 0.2% to 0.5%. At a drift ratio of 0.6% the maximum shear distortion demand, which was 0.011 rad generated in the interior joint indicating a critical damage in this joint and consequently, the distortion demand decreased progressively with the applied loading. Meanwhile, the distortion demand in the exterior joint exceeded that in the interior joint at a drift ratio of 1.2% and at a drift ratio of 1.6%, the deformation demand reached the upper limit of the critical damage, which was 0.015 rad. As reported by Calvi *et al.* [C6] the test was interrupted at this stage due to the significant strength reduction.

3.5 Comparison of Vecchio's Experimental and Analytical Results

3.5.1 Validation of Test Results

Deformation

In the test, column axial loads were first applied and followed with a series of progressively increased lateral forces. Since the axial loads were directly applied on the column tops, therefore significant beam bending moment did not develop initially. **Figure 3-27** shows the deformed shape under the combined loadings. It can be expected that beam behaviour was not dominated by gravity loads. In addition, a larger magnitude of column axial loads would result in an evident P- Δ effect, which could not be ignored in the analysis.

Figure 3-27 Deformed shape of Vecchio's frame under combined column axial and lateral loadings

Load versus Displacement Relationship

It was reported that for a lateral loading of 52.5 kN, flexure cracks were observed in the first storey beam followed by the flexural cracking at the base of the columns at a loading of 145 kN and as a result, the structure became softer obviously. At a loading of 287 kN, yielding in the top bars occurred shortly after the yielding in the bottom bars at the end of the first storey beam at a loading of 264 kN. This resulted in a perceptible drop in the lateral stiffness of the frame. Yielding and hinging at the base of the columns and at both the ends of the first storey beam occurred as the loading approached to 323 kN. At a loading of 329 kN, concrete crushing and hinging appeared evidently at the base of the columns. At the same time, yielding and hinging developed at both the ends of the top storey beam. The stiffness of the load-deformation relationship decayed rapidly but the frame demonstrated excellent ductility after achieving the ultimate loading.

The frame sustained an ultimate lateral loading of 332 kN at a drift ratio of 2.0%. The failure mechanism included ductile hinging at the ends of the beams and the base of the columns and the failure mode in each member was mainly flexural failure. The $P-\Delta$ effect had a significant influence on the bearing capacity of the structure, accounting for about 12 % of the total overturning moment.

Figure 3-28 presents the loads versus horizontal displacement relationships from analytical and test results. For a drift ratio of 0.4% to 1.5%, the analytical results showed a slightly higher load-carrying capacity than the test results at the same loading levels. As a consequence, the analytical frame behaved much stiffer than the test one, but the difference between them could be ignored. For a drift ratio of 1.5% to 2.5%, the experimental and analytical results were more comparable each other, and sustained their load-carrying capacities during this period. When the test frame was loaded into a drift ratio of 2.6%, strength degradation was observed. Up to the final displacement corresponding to a drift ratio of 3.8%, the measured load-carrying capacity of the test frame was 94% of the

maximum load-carrying capacity. However, in the analytical frame, the strength degradation was not found, and the load-carrying capacity increased progressively but only to a limited extent until the end of the analysis.

Figure 3-28 Comparison of loads versus horizontal displacement relationship between experimental and analytical results of Vecchio's frame

Figure 3-29 Hinge patterns and ductility demands in Vecchio's frame via finite element analysis

Damage and Plastic Hinges Development

Similar hinge patterns were found in the results from finite element analysis (see **Figure 3-29**). Beam hinges in the first storey beam occurred at a drift ratio of 0.83% and followed by the hinges generating in the second storey beam at a drift ratio of 1.67%. In addition, due to the uniform hoop spacing in the columns, column hinges at base sections could not be avoided. Full hinges developed in the base of columns at a drift ratio of 1.67%. However, no plastic hinges were found at other locations in the columns.

3.5.2 Discussion of Analytical Results

Only the exterior beam-column joints made up in Vecchio's test frame. In this study, the lower left-hand and lower right-hand joints are designated as Joint A and Joint B; the upper left-hand and upper right-hand joints are designated as Joint-C and Joint-D, respectively.

Joint Shear Stress

Figure 3-30 shows the nominal shear stresses in the joint panels via finite element analysis. Joint B behaved with a desired joint shear stress demand, which was much lower than the ACI 318 [A4] limit. The maximum shear stress was found in Joint B approximately at a drift ratio of 3.8%; which was 5.64 MPa or $1.03\sqrt{f'_c}$. Shear stresses in Joint-A presented a progressively increasing manner. At a drift ratio of 1.7%, the shear stress in Joint A was equal to that proposed in the ACI-ASCE 352 [A5] recommendation, and the maximum shear stress in this joint was about 9.26 MPa or $1.7\sqrt{f'_c}$, which was found at the end of the analysis.

The shear stresses in Joint C and Joint D from analyses were higher than that the ACI-ASCE 352 [A5] recommendation specified. Amongst them, shear stresses in Joint C remained approximately constant; whereas shear stresses in Joint D presented a slightly increasing manner. At the beginning of the analysis, the minimum shear stresses of Joints C

and D were $1.55\sqrt{f'_c}$ and $1.53\sqrt{f'_c}$, respectively and at a 2.5% drift ratio, all of them reached the maximum joint shear stress, which were 8.76 MPa or $1.6\sqrt{f'_c}$ and 9.34 MPa or $1.73\sqrt{f'_c}$, respectively. In the subsequent loading cycles, the shear stresses were slightly decreased in Joint D.

Figure 3-30 Nominal joint shear stresses of Vecchio's frame via finite element analysis

Diagonal Joint Compression Stress

Figure 3-31 illustrates the diagonal compression stresses in all the joint panels except Joint-D. The maximum diagonal compression stress obtained in Joint A at a drift ratio of 3.8% ($0.52 f'_c$) was slightly higher than the diagonal compression strength. Joints B and C remained at a low compression stress level during the analysis with the maximum diagonal compression stresses were $0.45 f'_c$ and $0.47 f'_c$, respectively. On the contrary, Joint D showed a higher magnitude of diagonal compression stresses at the beginning of analysis,

which was $0.52f'_c$. Until the end of the analysis, Joint D did not lose its diagonal compression strength by much.

Figure 3-31 Diagonal joint compression stresses of Vecchio's frame via finite element analysis

Diagonal Joint Tension Stress

Figure 3-32 shows the diagonal tension stresses in the joint panels via finite element analysis. In general, the diagonal joint tension stress behaviour was similar with that of the joint shear stress. On the first floor, Joint-B behaved with a desired diagonal tension stress with the maximum stress at approximately at a 3.8% drift ratio, which was about $0.16\sqrt{f'_c}$. Diagonal tension stresses in Joint A presented a constant increasing manner resulting in the maximum value being found at the end of the analysis, which was $0.42\sqrt{f'_c}$.

Joint-C and Joint-D were subjected to higher demands of the diagonal tension stresses, which remained constantly during the whole analysis. The maximum diagonal tension

stresses in Joint-C and Joint-D were reached at a drift ratio of 2.5%, which were $0.44\sqrt{f'_c}$ and $0.38\sqrt{f'_c}$, respectively.

Figure 3-32 Diagonal joint tension stresses of Vecchin's frame via finite element analysis

Tension Stresses in Joint Hoops

As mentioned in the previous section, all the beam-column joints except the lower-right one reached a high shear stress level, which significantly exceeded the ACI 318 [A2] limit on exterior joints. It is believed that the joint transverse reinforcement contributed significantly in carrying a large magnitude of the joint shear force.

In the test, all the joint cores were reinforced with the column hoops, which had the yield strength of 456 MPa. It can be expected that joint hoops with higher yield strength can afford tremendous joint shear stress without yielding. As a result, the frame retained its load-carrying capacity until a drift ratio of 4% without strength degradation, but the joint shear stress could be higher. Tension stresses in the joint hoops via finite element analysis

as shown in **Figure 3-33**. Although the stresses increased progressively with the loading run, the yield stress did not reach until the end of analysis. Use of high strength joint hoops can effectively resist joint shear forces and reduce the amount of joint hoops.

Figure 3-33 Tension stresses in the joint hoops of Vecchio's frame via finite element analysis

3.5.3 Discussion of Analytical Results of Modified Vecchio' Frame

3.5.3.1 Overall Behaviour

In order to investigate joint confinement effect on the performance of Vecchio's frame, a modified model was set up based on the original Vecchio's test frame. In this modified model, no joint hoops were provided in the all joint cores. From the loads versus horizontal displacement relationship curve, it can be seen that if the joint cores were not confined by the transverse reinforcement, the overall behaviour could only be satisfied before a drift ratio of 0.68% and after that the load-carrying capacity of the modified model became lower than that of the test frame. The analytical model reached its maximum load-carrying capacity at a drift ratio of 1.68% and then progressively lost its capacity. At a drift ratio of

2.7% the residual load-carrying capacity was only 80% of the maximum value. It is believed that at this moment the frame failed because the joint shear failure. **Figure 3-34** illustrates the loads versus horizontal displacement relationship for modified Vecchio's frame.

Figure 3-34 Comparison of loads versus horizontal displacement relationship between experimental and analytical results of modified Vecchio's frame

Figure 3-35 illustrates the hinge patterns and ductility demands of modified Vecchio's frame. Due to the absence of joint hoops, the modified frame presented different hinge patterns compared to the original frame. At a drift ratio of 0.83%, only column hinges were found at the base of each column. Beam plastic hinges were not formed until the frame was loaded to a drift ratio of 1.67%, when the beam plastic hinges occurred in the first storey beam. In addition, tension yield at the top face in the second storey beam and column were found at this stage. When the frame was loaded into a drift ratio of 2.6%, beam plastic hinges at both the top and bottom faces in the second storey beam were formed.

Figure 3-35 Hinge patterns and ductility demands in modified Vecchio's frame via finish element analysis

3.5.3.2 Joint Shear Behaviour

Joint Shear Stress

Comparing with the original frame, joint panels in the modified frame showed a different performance in terms of joint shear stresses as illustrated in **Figure 3-36**. The upper-right hand joint and joints on the first floor presented an identical manner with the maximum shear stresses being reached at a drift ratio of 1.68%. The maximum shear stress in Joint A, Joint B and Joint D was $1.6\sqrt{f'_c}$, $1.4\sqrt{f'_c}$ and $1.5\sqrt{f'_c}$, respectively and after that they could not resist the induced shear forces. Among these four joints, Joint C showed a different manner compared to the others. Shear stresses in Joint C remained at a low level indicating that it was not damaged in the early stage and could sustain a higher shear force until the completion of the test at a 3.8% drift ratio, where the maximum shear stress was reached by slightly exceeding the limited level recommended by ACI-ASCE 352. [A2].

Diagonal Joint Compression Stress

Figure 3-37 shows the diagonal joint compression stresses of modified Vecchio frame. Joints A to C were subjected to a desired diagonal compression stress demand. Up to the

final stage, the predefined diagonal compression strength was not reached. At a drift ratio of 2.5%, Joint D showed a higher diagonal compression stress demand with the maximum stress exceeding the limited compression strength of 16%, which was $0.58f'_c$.

Diagonal Joint Tension Stress

Joints B and C were subjected to lower diagonal tension stresses, where the maximum tension stress was lying on or below the recommended joint tension strength. Joints A and D reached their maximum tension stresses, which were $0.38\sqrt{f'_c}$ and $0.35\sqrt{f'_c}$, respectively at a drift ratio of 1.68% (see **Figure 3-38**).

Figure 3-36 Nominal joint shear stresses of modified Vecchio's frame via finite element analysis

Figure 3-37 Diagonal joint compression stresses of modified Vecchio's frame via finite element analysis

Figure 3-38 Diagonal joint tension stresses of modified Vecchio's frame via finite element analysis

3.6 Discussion of Joint Failure Modes of Analytical Frames

3.6.1 Joint Failure Model of Calvi's Frame

Since the beam longitudinal bars did not reach the yield strain during the test, the induced shear stresses in the joints were lower than what specified in the ACI-ASCE 352 [A5] recommendation on exterior joints. Although, the nominal joint shear stresses remained at a lower level during the experiment and finite element analysis, the extensive damage in the joint panels could not be prevented at early stages. A large magnitude of diagonal joint compression stresses indicates a joint failure mode which is mainly due to the diagonal concrete compression strut failure. Since the experimental and analytical frames were originally designed for resisting gravity-loads only, the joint panels were not reinforced with joint hoops and as a result, the induced shear forces would be independently carried by the diagonal concrete compression strut mechanism. If column axial loads had applied, the diagonal compression forces could have balanced to some extent. However, once the concrete tension stresses exceed the concrete tension strength, the diagonal tension forces, which are perpendicular to the failure plane, will result in the concrete compression strut failure. In the Calvi *et al.* [C6] study, this mechanism was defined as a shear hinge mechanism.

During the test, joint cracks occurred as early as reaching a loading level of 0.2% drift ratio. Through finite element analysis the extensive damage was found in both the interior and exterior joint panels on the first floor. The activity of a shear hinge mechanism modifies the global inelastic response and redistributes the deformation demand within the joint panel. The generation of the shear hinge mechanism reduces rotation demands in the columns. Consequently, a soft storey mechanism is avoided or at least postponed to a higher level of performance demand. On the other hand, the joint shear hinge mechanism will require a large magnitude of joint deformation demand, which may result in a sudden degradation with a loss in the load-carrying capacity of the whole frame.

3.6.2 Joint Failure Model of Vecchio's Frame

In the original Vecchio's frame, joint panels were subjected to large magnitudes of joint shear demand and diagonal joint tension stress demand. The joint hoops with the high yield strength effectively participate in joint shear resistance and provide confinement to the joint panel. As a result, a shear hinge mechanism is not evident at an early stage. Since the frame was lightly reinforced and a large magnitude of column axial loads was applied, the truss mechanism may be too weak to develop joint cracking. The critical parameter here is the diagonal tension stress in the joint panel rather than the shear stress level. Therefore, the joint failure mode can be ranked into Category 1 according to the definition of CEB [C2].

In the modified Vecchio's frame, the joint failure mode is still dominated by the joint shear stresses and diagonal tension stress. The joint shear mechanism postpones the formation of the beam hinges and results in the reduction of the system load-carrying capacity.

3.7 Summary

Finite element analysis for the tested frames reported by Calvi *et al* [C6] and Vecchio *et al* [V1] provided important information about the seismic performance of gravity-load-designed and lightly reinforced RC frames, which were inadequate to resist earthquake attacks. The conclusions from comparison between the results of test and finite element analyses are summarized as follows:

- (1) This chapter shows close agreement between the experimental and analytical results.

In general, the gravity-load designed RC frame could sustain its load-carrying capacity before a drift ratio of 2.0% was reached and after that strength degradation was found in both the experimental and analytical analyses. At an early stage, a large magnitude of joint shear demands cannot be prevented and joint shear failure is identified particularly in exterior joints.

- (2) Joint shear mechanism, which was found in the experimental investigation, is confirmed through finite element analysis. The joint shear hinge mechanism is mainly due to the joint shear failure, compression failure or tension failure. Due to the extensive joint damage occurring at an early stage, the global damage and hinge patterns are changed as a result.
- (3) In a gravity- load-designed RC frame, a combined mechanism rather than a pure soft-storey mechanism is expected during an earthquake. The development of beam or column flexural hinges and a soft-storey mechanism, if unable to be prevented, are postponed due to the occurring of shear hinges.
- (4) Modification of reinforcing details can improve joint behaviour to some extent. However, this technique cannot improve the global behaviour effectively. In particular, changing of joint reinforcing details will lead to both favourable and unfavourable effects on the overall response. If a joint panel is well confined, early joint damage can be prevented and therefore, the joint shear-carrying capacity can be enhanced. On the other hand, the joint deformation demand will be constrained which results in a large deformation demand in the adjacent member sections.

The analytical results, such as capacity curve, cracking patterns and hinge patterns matched the experimental results very well. However, some important issues such as bond slippage are not considered in the current analyses. Therefore, the behaviour of lightly reinforced beam-column joints should be investigated in more details.

Chapter 4

Experiment Setup and Test Methods

4.1 Introduction

In Chapter 2, design criteria and typical reinforcing details required for gravity-load-designed or lightly reinforced concrete frame structures were summarized. Comparing with the modern seismic design provisions, reinforcing details of the reinforced concrete frame buildings designed conforming to a non-seismic design code are inadequate for seismic resistance. This may cause the structures failure during a moderate earthquake action. In Chapter 3, experimental studies performed by Calvi *et al.* [C6] and Vecchio *et al.* [V1] were reviewed and the analytical studies were carried by finite element approach. Joint shear failure, which attributed to the joint shear hinge mechanism, was found and followed by a combined mechanism in these studies. Hence, it is necessary to carry out more experimental investigations to provide further information on the seismic behaviour of beam-column joints with non-standard reinforcing details. In this research, the experimental studies are to be carried out with an emphasis on examining the seismic behaviour of beam-column joints designed with seismic deficiencies.

The experimental work in this research program consists of two series of experiments. The first series of the tests is conducted to study the behaviour of non-seismically detailed interior beam-column joints subject to lateral forces with consideration of slab effects. The second series of the tests is aimed at investigating the seismic behaviour of wide beam-column joints. In the first series, five conventional beam-column joint specimens were tested and, in the second series six wide beam-column joint specimens including three

interior and three exterior connections were tested. This chapter describes the design, construction, and test methods of these beam-column joint specimens. The measurement to obtain the applied forces, displacement, and strains are also introduced.

4.2 Test Specimens Design

4.2.1 Modelling Criteria and Design Forces

The structural prototype was framed by a conventional beam or wide-beam frames in both longitudinal and perpendicular directions. The clear span and floor height of the prototype frame were 5.4m and 3.6m, respectively. The test specimens represented the typical interior connections and exterior tee connections of these reinforced concrete frames. The interior joint specimens contained a column and two main beams in the longitudinal direction. In addition, a slab and transverse beam would be involved if available. However, only one main beam was included in the exterior joint specimens. Moreover, as mentioned above, the exterior joint connections were only investigated in the second series of the experiments.

Beam and column cross-sections were designed based on the most severe stresses produced by the load combinations required by the BS 8110 [B3] code. These load combinations are:

- $1.4g_k + 1.6q_k$
- $1.4g_k + 1.4w_k$
- $1.2g_k + 1.2q_k + 1.2w_k$
- $1.0g_k + 1.4w_k$

where, g_k and q_k is the distributed dead load and live load. In this study, they were taken as 6.0 kN/m² and 5.0 kN/m², respectively. Wind loads are also considered in the BS8110

[B3] design code requirement. The wind load at the floor level of the frame w_{ki} was calculated as follows:

$$W_{ki} = C_f q_i A_i \quad (4-1)$$

where

$$q_i = 0.613 V_s^2 \quad (4-2)$$

$$V_s = V S_1 S_2 S_3 \quad (4-3)$$

where

C_f : Force coefficient, taken as 1.1 in this study

A_i : Frontal area at the i^{th} floor

V : Basic wind speed defined as the 3-sec gust with 50 years of a return period, 10m above ground in an open area with no obstructions, taken as 32 m/s in this study.

S_1 : Correction factor to account for variations in the wind environment due to topography, taken as 1.0 in this study.

S_2 : Correction factor to account for general ground roughness, building dimension and the height above ground.

S_3 : Correction factor to account for the statistical characteristics of the wind, taken as 1.0 in this study

Considering the robustness, BS 8110 [B3] requires that ‘all the buildings should be capable of resisting a notional design ultimate horizontal load applied at each floor or roof level simultaneously equal to 1.5% of the characteristic dead weight of the structure between mid-height of the storey below and either mid-height of the storey above or the roof surface’. In this study the design ultimate wind loads for the frame was less than this

notional load, and the effect of this load was small comparing to the effect of the design ultimate vertical live and dead loads. Therefore, the frame was designed only with consideration of gravity loads.

The test specimens were cut at the mid-height of the columns and at the mid-span of the beams where were assumed contra-flexural points for the bending moment diagram due to the lateral forces (see **Figures 4-1** and **4-2**). In order to match the dimensions of the test frame, the overall dimensions of these specimens were scaled approximately to 3/4 of the original connections in the buildings. So that the height of the column was 2725 mm and the span of the beam was 4000 mm, both measured from the centre of the pins in the test fixture.

Figure 4-1 Typical deformation of frame under gravity and lateral load

Figure 4-2 Test specimen configurations in prototype frame

4.2.1 Description of Conventional Beam-Column Joints

In the first series of the experiments, a total of five specimens were divided into two groups. Group one consisted of three beam-column joints, which were designated as AL1, AS1 and MAS1. Among them, Specimen AL1 was a pure beam-column joint, which consisted of two main beams and a column only. Here, “AL” stands for “As built lightly reinforced beam-column joint unit.” Specimen AS1 was a full beam-column joint since there was a slab and a transverse beam cast into the connection, therefore “AS” stands for “As built lightly reinforced beam-column connection with a slab.” The configuration of Specimen MAS1 was developed based on that of Specimen AS1. However, some modifications were given to investigate the effect of these variables on the response of the connection. Details of the modifications on Specimen MAS1 would be described in the following sections. In group two, there were two specimens. Herein and hereafter, the pure beam-column specimen was labeled as AL2 and the specimen with a slab was labeled as AS2.

Referring to **Figure 4-2**, specimens in group one are considered as the connections that were picked up from a weak direction of the moment resisting frame because the columns of Specimens AL1, AS1, and MAS1 could be loaded in bending about their weak bending axes. Conversely, columns of Specimens AS2 and AL2 would be able to be loaded in bending about their strong bending axes as such Specimens AL2 and AS2 represent the connections of a strong direction of the moment resisting frame.

Overall dimensions and reinforcing details of conventional beam-column joint specimens are shown through **Figure 4-3** to **Figure 4-6**. Reinforcing details of these specimens were followed the current design practice based on the BS8110 [B3] design code. Beams were reinforced with 2T20 as bottom bars and a combination of 2T20 and 1T16 was used as top bars giving a beam longitudinal top steel ratio at 1.13% and a bottom steel ratio at 0.86%. Beam bars were continuous throughout the whole beam span. A total of 8T20 longitudinal

bars, which were lapped above the floor level, were used in columns resulting in a column steel ratio at 3.14%.

The transverse reinforcement required for resisting shear, confining the concrete, and preventing premature buckling of the longitudinal bars in the beams and columns, was detailed according to the requirement of the BS8110 [B3] design code. Beams were reinforced with R10 hoops at 150 mm spacing and columns were reinforced with R10 hoops at 200 mm spacing. There was no transverse reinforcement put into the joint core except for Specimen MAS1.

The longitudinal reinforcing details of Specimen MAS1 were essentially identical to the reinforcing configuration of Specimen AS1. Modifications were concentrated on the details of the transverse reinforcement in the beams, column, and joint regions. In Specimen MAS1, hoops' end was bent with 135 angles rather than 90 angles which were used in other specimens. Within the potential hinge regions, which equal to 1.5 times of the beam depth or column depth, the column and beam hoop spacing was reduced from 150 mm to 100 mm, meanwhile the joint core was confined by using three sets of column hoops.

Figure 4-3 Dimensions and reinforcing details of Specimens AL1 and AS1

Figure 4-4 Dimensions and reinforcing details of Specimens AL2 and AS2

Figure 4-5 Dimensions and reinforcing details of Specimen MAS1

Concerning the flexural strength ratio, in the case of the columns were loaded about their strong axes, the columns were designed to have an ideal flexural strength of 1.26 times the beam's flexural strength. In the contrary, when the columns were going to be loaded about their weak axes, this ratio became 0.68 leading to a “strong beam-weak column” configuration. In order to calculate this ratio, it was assumed that plane sections retain plane and an equivalent rectangular concrete stress block was taken.

If slab bars were considered fully effective in resisting negative bending moments, the column to beam flexural strength ratio for the full connection specimens was reduced to approximately 1.09 (for the strong axes case) and 0.58 (for the weak axes case), respectively. Since no conclusive data are available on the extent of the participation of the slab bars in resisting the negative bending moments, in this research, the entire width of the slab is conservatively assumed to be effective in calculating the negative flexural strength of the beams.

Figure 4-6 Section details of Specimens AL1, AL2, AS1, and AS2

4.2.2 Description of Wide Beam-Column Joints

For the purpose of description in this and subsequent chapters, the interior wide beam-column joint specimens are marked with IWB-1, IWB-2, and IWB-3. Similarly, the exterior wide beam-column joint specimens are marked with EWB-1, EWB-2, and EWB-3. The overall dimensions of these specimens are shown in **Figures 4-7 to 4-11**.

The main beam section of these specimens was 800 mm×300 mm and the column cross-section was 900 mm×300 mm. In Specimens IWB-1 and IWB-3, columns were orientated such that they would be able to bend about their strong axes, which would lead to a beam width to column width ratio b_w/b_c at 2.67. A transverse beam with a cross-section of 800 mm×300 mm was cast into the joint core of these two specimens. Since the depth of the transverse beam was identical to that of the main beam, therefore, it was also a wide

beam. Conversely, Specimen IWB-2 could not be strictly classified as a wide beam because its column width was slightly wider than that of the beam giving a beam width to column width ratio at 0.89 (see **Figure 4-7**). Since all the beam longitudinal bars of Specimen IWB-2 could be anchored in the column core, a transverse beam was not necessary in this specimen.

(a) Interior wide beam-column Specimens IWB-1,2 and 3

(b) Exterior wide beam-column Specimens EWB-1, 2 and 3

Figure 4-7 Wide beam-column test specimens with different configurations

Section dimensions and reinforcing details of the exterior wide beam-column specimens (EWB-1 to EWB-3) were essentially identical to the corresponding interior wide beam-column specimens (IWB-1 to IWB-3) except for only the one-side beam being framed into these specimens. Beam longitudinal bars within an exterior beam-column joint were bent into the joint core at a certain length to prevent a bond slip due to the insufficient bar anchoring.

3T22 plus 6T20 bars were used as the top flexural bars for the all wide beam-column joint specimens, the bottom bars were 3T20 plus 6T16, and the top and bottom longitudinal steel ratios were 1.43% and 1.0% respectively. For the exterior Specimens EWB-1, EWB-3 and interior Specimens IWB-1, IWB-3, 3T22 top bars and 3T20 bottom bars were placed inside the column core, which indicated that 37.7% of the top bars and 43.8% of the bottom bars were anchored in the column core. Columns were uniformly reinforced with 14T25 bars, which gave a column reinforcing ratio being of 2.54% for the all specimens. The longitudinal bars of the transverse beam consisted of 5T20 top bars and 5T22 bottom bars.

The transverse reinforcement used in the wide beams and columns were provided by sets of overlapped R10 hoops, which were placed at every 100 mm interval within the potential hinge regions and were relaxed to 200 mm spacing in the remainder ranges. Each set of the beam transverse reinforcement consisted of three rectangular hoops and each set of the column transverse reinforcement had two rectangular hoops. Three sets of overlapped hoops with the same type as the column transverse reinforcement were used as horizontal joint shear reinforcement, which were fabricated with T16 deformed bars.

In Specimens IWB-1, IWB-3, EWB-1, and EWB-3, the column depth to beam bar diameter ratio ranged from 41 to 56.3. Obviously, these values were much higher than the recommended value being of 20 by the ACI 318 [A2] design code and imply a satisfied bond condition. For Specimens IWB-2 and EWB-2, due to the different column orientation, these values changed to 13.6 for the T22 bar and 15 for the T20 bar, which were inferior to the ACI 318 [A2] design code. In all wide beam-column connections, the beam depth to column bar diameter ratio was 12, which was much lower than the ACI 318 [A2] limit.

Figure 4-8 Dimensions and reinforcing details of Specimens IWB1 and IWB3

Figure 4-9 Dimensions and reinforcing details of Specimen IWB2

Figure 4-10 Dimensions and reinforcing details of exterior wide beam-column joint specimens

(a) Beam section A-A

(b) Column section B-B

(c) Column section C-C

(d) Beam-column joint

Figure 4-11 Section details of wide beam-column joint specimens

4.3 Test Setup

The test setup is shown in **Figures 4-12** and **4-13**. The boundary conditions are appropriate for the lateral displacement, considering the assumption of zero bending moments at the mid-span of the beams and the mid-height of the column for a frame under lateral forces. The column was pinned to the floor while the top end is connected to an actuator. The in-plane horizontal loadings could be applied on the column top by a double acting 300 kN capacity hydraulic jack. A strain gauged load cell was placed between the jack and a link component connected to the top hinge.

The beams were connected at the mid-span positions through pin-ended steel struts. This connection allows free horizontal movement and rotation of the beams but there was no vertical displacement allowed in the specimen similar to that in a joint of a frame under cyclic lateral forces. For all the experiments in this study, there was no axial loading applied on the columns.

Figure 4-12 Test setup

Figure 4-13 Photograph of test setup

4.4 Test Procedure and Loading Sequence

The loading sequence adopted in this test program followed a typical quasi-static test sequence, which used for many years by other researchers [H1]. It is believed that the probable seismic resistance of a sub-component evaluated following the simple test sequence is able to provide a satisfactory behaviour during a real seismic event [H1].

When the ductility controlled loading manner was concerned, the specimens would be initially loaded with cycles of the elastic loading run up to $0.75 P_y$, where P_y is the theoretically calculated load-carrying capacity. It was calculated based on the flexural strength of the critical component, which was determined by using measured material properties. In order to determine the yield displacement Δ_y , the lateral displacement at the peak of the loading cycle $0.75 P_y$ was firstly recorded and averaged and then the average value of $\Delta_{0.75}$ was available, the reference yield displacement Δ_y of the sub-assemblages can be obtained by linear extrapolation as:

$$\Delta_y = \frac{4}{3} \Delta_{0.75} \quad (4-4)$$

The definition of reference yield displacement is expressed graphically in **Figure 4-14**. Once the yield displacement is found, the target lateral displacement of the test specimen in terms of displacement ductility factor can be established and expressed in the following equation:

$$\mu_{\Delta} = \frac{\Delta}{\Delta_y} \quad (4-5)$$

where, Δ is the applied lateral displacement. The complete loading sequence by means of ductility control is shown in **Figure 4-15(a)**.

Figure 4-14 Definition of the yield displacement of test unit

After the first specimen tested, it was found that there was a significant gap between the measured yield displacement and the predicated value. This might impute to the overestimating of the specimen's capacity by ignoring of the early damage and without consideration of bond slip. Furthermore, due to the non-typical dimensions of the wide beam-column connections, the strength and initial stiffness were hard to be anticipated. Therefore, in the remainder experiments, the lateral loads were applied in terms of a drift

controlled rather than a ductility controlled manner. During the period of the test, a series of lateral displacements were imposed corresponding to a drift ratio from 0.5% up to 5.0%. (see **Figure 4-15(b)**)

(a) Loading sequences in terms of load and displacement control

(b) Loading sequences in terms of drift control

Figure 4-15 Loading histories in the test

4.5 Instrumentation

4.5.1 Observation of Cracking Development

All cracks observed in each test specimen were marked on the white painted concrete surfaces. In order to capture the development of the cracking, photographs were taken at the peak of each loading cycle and other relevant stages. The crack width in the joint and critical regions of the test specimens was measured by using a crack magnifier with a 0.02 mm division at the peak of each loading cycle.

4.5.2 Measurement of Loads

An electrical resistance strain gauge load cell was used to measure the applied loads from the hydraulic actuator. This load cell was designed and manufactured for measuring the tensile or compressive force applied at the tip of the column. The location of the load cell is shown in **Figure 4-12**.

4.5.3 Measurement of Displacement

External deformation of the specimens can be monitored by measuring the displacement. Two ranges of displacement are involved, which are a lower-level displacement such as a diagonal displacement in the joint, and a large-level displacement such as column top displacement at the level of the loading applied. In order to capture each range of displacement precisely, three types of LVDT (transducers) were used to measure the different displacements. The arrangement of transducers for measuring the displacement was identical for all specimens.

Figure 4-16 illustrates the distribution of these transducers. A transducer with a distance capacity being of 300 mm was placed at the tip of the column at the input-level of the lateral loads to measure the top displacement of the column; the other one with a same capacity was placed at a further end of the beam to measure rigid body movement. Two

transducers with a distance capacity being of 100 mm were placed vertically on the bottom of the beams to measure vertical displacement. The positions of this measurement were determined in which the maximum vertical displacement on the beams was expected. One more transducer with a distance capacity being of 100 mm was used to measure the lateral displacement on the level of the bottom end of the column.

Figure 4-16 Layout of measurement for external deformation

4.5.4 Measurement of Average Curvature of Beams and Columns

A total of twelve pairs of transducers with a maximum distance capacity being of 50 mm were attached to an embedded threaded rod. They were able to obtain the average curvature of the column and beam, which significantly contributed to the total displacement. The measurement was taken on the critical regions where plastic hinges were potentially expected.

The average rotation over a region also could be obtained through measuring the displacement by using the top and bottom transducers. With reference to **Figure 4-17**, the average curvature over a certain region in the beam can be expressed as follows:

$$\theta_{b,i} = \frac{{}_t\delta_i - {}_b\delta_i}{h_i} \quad (4-6)$$

$$\phi_{b,i} = \frac{\theta_{b,i}}{s_i} \quad (4-7)$$

where,

- ${}_t\delta_i$: Top displacement measured over a certain region in the beam
- ${}_b\delta_i$: Bottom displacement measured over a certain region in the beam
- h_i : Distance between the top and bottom transducers in a certain region
- $\theta_{b,i}$: Rotation measurement over a certain region in the beam
- $\phi_{b,i}$: Average curvature over a certain region in the beam
- s_i : Gauge length of a certain region in the beam

Similarly, the average curvature over a certain region in the column can be expressed as below:

$$\theta_{c,i} = \frac{{}_l\delta_i - {}_r\delta_i}{h'_i} \quad (4-8)$$

$$\phi_{c,i} = \frac{\theta_{c,i}}{s'_i} \quad (4-9)$$

where

- ${}_l\delta_i$: Left displacement measured over a certain region in the column
- ${}_r\delta_i$: Right displacement measured over a certain region in the column
- h'_i : Distance between the left and right transducers in a certain region
- $\theta_{c,i}$: Rotation measurement over a certain region in the column

$\phi_{c,i}$: Average curvature over a certain region in the column

s'_i : Gauge length of a certain region in the column

It should be noted that the beam and column shear deformation was not evaluated in each of the test.

Figure 4-17 Evaluation of fixed-end rotation, curvature and flexural deformation (after Hakuto)

4.5.5 Measurement of Average Joint Shear Distortion

Due to the three dimensional natures of the specimens, joint shear deformation is hard to be measured directly. Therefore, among these specimens, only Specimens AL1 and AL2 are allowed to evaluate the joint shear deformation. In these two test units, the transducers were placed diagonally in the joint panel to measure the average shear distortion and to estimate the expansion of the joint. **Figure 4-18** shows an assumption of the deformed shape of the joint core concrete and the average joint shear distortion γ_j is given as:

$$\gamma_j = \gamma_1 + \gamma_2 = \frac{\delta - \delta_j}{2l_j} \left(\tan \alpha_j + \frac{1}{\tan \alpha_j} \right) \quad (4-10)$$

where, δ and δ_j are the changes of the diagonal lengths; l_j is the initial length of the diagonal in the joint core.

The expansion of the joint core was estimated by measuring the diagonal joint displacement and was defined as an average value of the diagonal joint displacement, which is $(\delta + \delta_j)/2$. Joint expansion can be used as an index to gauge the failure of the joint core concrete.

Figure 4-18 Assumption of joint core deformed shape (after Hakuto)

The measurement did not make for the remainder specimens, so that the calculation of the joint deformation for these specimens was impossible as a result.

4.5.6 Measurement of Reinforcing Bars' Strains

To measure reinforcement strains, two sizes of post-yield strain gauges were used, which have a gauge length at 5 mm and 3 mm, respectively. These strain gauges have been pre-attached with three 10 meter long parallel vinyl-insulated lead wires.

In order to attach these strain gauges, the bar surface deformation in the selected regions must be removed and polished. After that, the strain gauges were attached to the bars by using cyanoacrylate and followed by the application of a layer of RTV-3140 silicone. The second layer of silicone was applied at 24 hours later and finally, a piece of ABM-75 protective compound was pasted on to provide protection against the surrounding concrete.

It should be noted that in the first test, strain gauges were attached symmetrically at each selected position and therefore, an average strain value was obtained as a result. In the second test, due to a lot of bars and limited strain gauges, symmetrical strain gauging was not advisable. Instead, the strain gauge arrangement was designed carefully to obtain a useful record. The strain gauge distribution will be described in details in the later chapters.

4.6 Components of Horizontal displacement

The horizontal displacement of the test specimens are composed of varied deformation coming from the beam, column, and joint with some unexpected and unmeasured factors being included. This section describes varied deformation contribution to the total horizontal displacement.

4.6.1 Beam Deformation

Flexural Deformation

Flexural deformation of the beam was obtained from the rotation of each region in the beam, which was measured from a pair of the top and bottom transducers. Referring to **Figure 4-17**, the flexural deformation of the beam can be expressed as follow:

$$\delta_{bf} = \sum \theta_{b,i} (l'_b - x_i) \quad (4-11)$$

where

$\theta_{b,i}$: Rotation measured over a certain region in the beam

l'_b : Distance from the column face to the center of the beam end pin

x_i : Distance from the column face to the center of the region i

Therefore, the horizontal displacement at the column top due to the beam flexural displacement is presented by:

$$\Delta_{bf} = \frac{h}{l} \delta_{bf} \quad (4-12)$$

where

Δ_{bf} : Equivalent horizontal displacement due to the beam flexural deformation

h : Storey height or vertical distance between the column end pins (=2725mm in this research)

l : Beam span or horizontal distance between the beam end pins (=4000mm in this research)

Fixed-end Rotation

A pair of transducers located near to the column faces can be use to estimate the fixed-end rotation of the beam. Referring to **Figure 4-17**, the beam fixed-end rotation can be expressed as follow:

$$\theta_{bfe} = \frac{{}_t\delta_1 - {}_b\delta_1}{h_1} \quad (4-13)$$

where

${}_t\delta_1$: Top displacement measured at the fixed end interface

${}_b\delta_1$: Bottom displacement measured at the fixed end interface

h_1 : Distance between the transducers at the fixed end interface in the beam

The deformation due to the beam fixed-end rotation can be obtained as:

$$\delta_{bfe} = \theta_{bfe} l'_b \quad (4-14)$$

where

θ_{bfe} : Beam fixed-end rotation

The horizontal displacement at the column top due to the beam fixed-end rotation can be expressed by:

$$\Delta_{bfe} = \frac{h}{l} \delta_{bfe} \quad (4-15)$$

where

Δ_{bfe} : Equivalent horizontal displacement due to the beam fixed-end rotation

4.6.2 Column Deformation

Flexural Deformation

Column flexural deformation contributes to the horizontal displacement can be expressed as follow:

$$\Delta_{cf} = \sum \theta_{c,i} (l'_c - y_i) \quad (4-16)$$

where

$\theta_{c,i}$: The rotation measurement over a certain region in the column

l'_c : The distance from the beam face to the column end pin

y_i : The distance from the beam face to the center of the region i

Fixed-end Rotation

The horizontal displacement due to the column fixed-end rotation can be obtained by using the same way used for the beam as:

$$\Delta_{cfe} = \theta_{cfe} l'_c \quad (4-17)$$

$$\theta_{cfe} = \frac{{}_l\delta_1 - {}_r\delta_1}{h'_1} \quad (4-18)$$

where

θ_{cfe} : Column fixed-end rotation

${}_l\delta_1$: Left displacement measured at the fixed-end interface

${}_r\delta_1$: Right displacement measured at the fixed-end interface

h'_1 : Distance between the transducers at the fixed-end interface in the column

4.6.3 Joint Deformation

The average joint shear distortion contributes the horizontal displacement can be referred to **Figure 4-17** and is expressed as below:

$$\Delta_j = \gamma_j \left(h - h_b - \frac{h}{l} h_c \right) \quad (4-19)$$

where

γ_j : Joint shear distortion

h_b : Beam depth

h_c : Column depth

4.7 Material Properties

Both the beam and column longitudinal bars were deformed bars with lower yield strength being of 460 MPa. The diameter of the longitudinal bars varied from 16 mm to 25 mm. R10 plain round bars with lower yield strength being of 250 MPa were used as transverse reinforcement in the all test units. T16 deformed bars were also used as joint transverse reinforcement for the wide beam-column joint specimens.

Steel samples cut from straight bars or hoops were tested monotonically in a 1000 kN Avery Universal Testing Machine. The test results showed that, in the first batch of steel specimens, the yield strength of the T16 steel bars was much higher than the predefined yield strength. However, the yield strength of the T20 bars was almost the same as the predefined value (see **Figure 4-19**). In the second batch, in which the steel specimens were cut from the bars of the wide-beam units, all the test results of the T16 to T25 steel specimens were higher than the predefined yield strength. R10 steel samples were not provided by the supplier, therefore, the test results could not be obtained. **Table 4-1** tabulates the test results of the steel samples.

Three 150×150×150 mm concrete cubes and three 150 mm diameter ×300 mm high concrete cylinders were prepared for each specimen. These units were tested to determine the compression strength of the concrete just before the test of the specimens. In the first series of the tests, all specimens were cast by using C40 concrete. In the second series of the tests, concrete with designed compression strength being of 70 MPa (C70) was used for Specimens IWB-1, IWB-2, EWB-1, and EWB-2. For the purpose of investigation the effect of concrete strength on the behaviour of the specimen, Specimens IWB-3 and EWB-3 were cast by using C40 concrete. The test results are showed in **Table 4-2**.

Figure 4-19 Strain versus stress relationship for steel specimens picked up from conventional beam-column joint specimens

Table 4-1 Test results of steel specimens

	Reinforcement	Diameter (mm)	Design yield stress (MPa)	Average stress at yield (MPa)	Average stress at failed (MPa)
Conventional Beam-Column Joints	R10	10	250	311	433
	T16	16	470	499	613
	T20	20	470	473	622
Wide Beam-Column Joints	R10	10	250	-	-
	T16	16	470	504	613
	T20	20	470	535	621
	T22	22	470	543	678
	T25	25	470	514	616

Figure 4-20 Strain versus stress relationship for steel specimens picked up from wide beam-column joint specimens

Table 4-2 Test results of concrete specimens

Units	Design Concrete	Age of Concrete (days)	f_c (MPa, Cube)	f'_c (MPa, Cylinder)
AL1	C40	94	56.4	31.7
AS1	C40	53	58.6	31.4
MAS1	C40	65	56.4	31.2
AL2	C40	34	55.8	34.6
AS2	C40	58	56.5	35.8
IWB-1	C70	-	79.8	64.1
IWB-2	C70	-	82.4	66.0
IWB-3	C40	-	56.9	47.9
EWB-1	C70	-	75.3	64.0
EWB-2	C70	-	75.2	64.9
EWB-3	C40	-	54.2	45.2

4.8 Prediction of Theoretical Yield Displacement and Initial Stiffness of the Test Specimens

Before each test, the theoretical yield displacement, $\Delta_{y,th}$, and the initial stiffness, K_{th} , of each test specimen were calculated. It was assumed that once the theoretical flexural strength was reached, bars at critical sections yielded simultaneously. At this moment the theoretical lateral load, P_{th} of the test specimen was developed. As mentioned earlier, when evaluating the theoretical strength of beam-column specimens with a slab, it was assumed that the slab bars in Specimens AS1, MAS1 and AS2 were fully effective in calculating specimen capacity. The theoretical elastic deformation of the beam and the column of each test specimen due to the flexure and shear were predicted as follows [H1]:

$$\delta_{b,th} = \delta_{b,fl,th} + \delta_{b,s,th} = \frac{V_b l_b^3}{3E_c I_e} + \frac{V_b f l_b}{0.2E_c b_b h_b} \quad (4-20)$$

$$\delta_{c,th} = \delta_{c,fl,th} + \delta_{c,s,th} = \frac{V_{col} l_c^3}{3E_c I_e} + \frac{V_{col} f l_c}{0.2E_c b_c h_c} \quad (4-21)$$

where $\delta_{b,fl,th}$ and $\delta_{c,fl,th}$ are the theoretical flexural deformation of the beam and column, $\delta_{b,s,th}$ and $\delta_{c,s,th}$ are the theoretical shear deformation of the beam and column, V_b and V_{col} are the beam and column shear forces at the development of the theoretical lateral load of the test specimen, b_b and b_c are the widths of the beam and column, f is the shape factor taken as 1.2, E_c is the elasticity modulus of the concrete and I_e is the effective moment of inertia of either the beam or the column.

Taking into account of the effect of concrete cracking on the stiffness in the beam and column, the effective moment of inertia was taken as $0.5I_g$, in which I_g is the moment of inertia based on the uncracked gross concrete area. Besides that, the shear deformation of the cracked member was estimated as twice of the uncracked member.

The elastic deformation contribution of the beam and column to the equivalent horizontal displacement at the top of the column could be given as:

$$\Delta_{b,th} = \frac{h}{l} \delta_{b,th} \quad (4-22)$$

$$\Delta_{c,th} = \delta_{c,th} \quad (4-23)$$

where $\Delta_{b,th}$ and $\Delta_{c,th}$ are the theoretical horizontal displacement due to deformation of the beam and column, respectively.

Deformation of the beam-column joint due to shear distortion was assumed to contribute to the total horizontal displacement by 20%. So that, the horizontal displacement due to the joint shear distortion, $\Delta_{j,th}$ was expressed as:

$$\Delta_{j,th} = 0.2\Delta_{y,th} \quad (4-24)$$

The theoretical initial yield displacement, $\Delta_{y,th}$, was given by the summation of all the theoretical horizontal displacement, which was:

$$\Delta_{y,th} = \Delta_{b,th} + \Delta_{c,th} + \Delta_{j,th} \quad (4-25)$$

And, the theoretical initial stiffness, $K_{theoretical}$, derived from below:

$$K_{th} = \frac{P_{th}}{\Delta_{y,th}} \quad (4-26)$$

Table 4-3 Theoretical load-carrying capacity, yield displacement and initial stiffness

	Specimen	P_{th} (kN)	$y_{,th}$ (mm)	K_{th} (kN/mm)
Conventional Beam-Column Joints	AL1	60.6	12.5	4.8
	AS1	63.1/97.3	12.6	5.0
	MAS1	63.1/97.3	12.6	5.0
	AL2	63.9	6.75	9.5
	AS2	66.7/102.7	6.54	10.2
Wide Beam-Column Joints	IWB-1	249	5.3	47
	IWB-2	207	6.2	33.4
	IWB-3	244	6.2	39.4
	EWB-1	+170.5/-123.6	10	17.1/12.4
	EWB-2	+144.4/-103	9.2	15.7/11.2
	EWB-3	+170/-122	9.7	17.5/12.6

Chapter 5

Test Results of Lightly Reinforced Beam-Column Joints

5.1 Introduction

This chapter examines the seismic behaviour of five lightly reinforced beam-column joints which were designed to BS8110 [B3]. For the ease comparison, subassemblages are divided into two groups according to their geometric orientation. Group one includes three specimens, AL1, AS1 and MAS1, which present the interior beam-column joints of a weak direction of the moment resisting frame as defined in Chapter 4. Specimens, AL2 and AS2 present the connections from a strong direction of the moment resisting frame. Other variables include the presence of a slab and the modifications of reinforcing details.

The response of each beam-column specimen is evaluated on either a global or local level. The global behaviour is discussed in terms of cracking patterns, hysteretic response, strength and stiffness degradation and the contribution of the component deformation to the lateral displacement. Moreover, the global behaviour is studied through the evaluation of the joint shear strength and shear distortion. The local behaviour is investigated through the measured strains in the longitudinal bars of the column and the beam. In addition, the observed behaviour of these specimens is discussed with regards to the effect of the column to beam flexural strength ratio, the effect of slab participation and the effect of joint confinement to the performance of the beam-column joint. Finally, finite element analyses are carried out to provide additional information on the seismic behaviour of the joints, which could not be measured in the tests.

5.2 Test Results of Specimens AL1, AS1 and MAS1

5.2.1 Specimen AL1

As mentioned early, Specimen AL1 was a one-way beam-column joint selected from the weak direction of the moment resisting frame. The geometries and reinforcing details have been described in Chapter 4. The typical features of this specimen are summarized as follows:

- (1) Column longitudinal bars were just lap spliced above the floor level;
- (2) Column ends as well as beam ends were lightly reinforced;
- (3) No transverse reinforcement was provided within the joint core.

In this specimen, the ratio of the column to beam flexural strength was 0.68, and thus plastic hinges were expected to form in the column and led to a strong beam-weak column structural configuration. The ratio of the column depth to beam bar diameter was $h_c / d_b = 10/1$ and the ratio of the beam depth to column bar diameter was $h_b / d_c = 20/1$.

Based on the column hinge mechanism, the theoretical capacity of Specimen AL1 was 60.6 kN in either loading direction. Following sections report the test results of the specimen.

5.2.1.1 General Observation

Figure 5-1 illustrates the cracking patterns of Specimen AL1 at different storey drift levels. When the specimen was loaded to a storey drift ratio of $\pm 0.5\%$, flexural and shear cracks initiated in the beams and column and at the same time, a few of small cracks were observed within the joint core.

When the specimen was loaded to a storey drift ratio of 1.0%, some diagonal tension cracks appeared in the joint core. In the first cycle of the storey drift ratio of 2.0%, the diagonal tension cracks kept increasing with one dominant crack opening widely. Bond splitting

cracks along the column bars extended and connected to the diagonal tension cracks in the joint panel. The specimen was finally loaded at a storey drift ratio of 3.7% with the load-carrying capacity dropping slightly at this stage.

Figure 5-1 Observed cracking of Specimen AL1 at each selected storey drift level

(a) Diagonal cracks in the joint region

(b) Concrete crush in the column compression zone

Figure 5-2 Photographs of Specimen AL1 after testing

In the joint core, concrete spalling due to the opening of crossed diagonal tension cracks was observed. In the column compression zone, concrete was crushed and followed by spalling. **Figure 5-2** presents photographs of Specimen AL1 after the test.

5.2.1.2 Hysteretic Behaviour

The storey shear force versus horizontal displacement relationship is shown in **Figure 5-3**. The theoretical capacity was not reached until the end of the test with the maximum capacity being obtained at a displacement of 37 mm corresponding to a storey drift ratio of 1.4%. Compared with the predicted yield displacement of Specimen AL1, this displacement was much higher, which was about three times the first yield displacement of 12.5 mm predicted by the theoretical approach. The maximum capacities of Specimen AL1 were 53 kN in both loading directions

Figure 5-3 Storey shear force versus horizontal displacement relationship for Specimen AL1

After the loading to a storey drift ratio of 1.5%, the hysteretic loops pinched significantly due to the severe bond deterioration along the beam and column bars as well as the joint

diagonal tension cracking. Consequently, the extensive of joint diagonal tension cracks caused the severe strength and stiffness degradation. At the end of the test, the residual load-carrying capacity of the specimen was about 37 kN in either loading direction and therefore, the behaviour of the test specimen was dominated by the response of the most damaged beam-column joint. **Figure 5-4** illustrates the storey shear force versus joint shear distortion relationship.

Figure 5-4 Storey shear force versus joint shear distortion relationship for Specimen AL1

5.2.1.3 Joint Behaviour

In the previous section, the joint damage has been discussed in terms of the opening diagonal tension cracks in the joint core. In this section, joint deformation is further evaluated in terms of the joint distortion and expansion with the addition that the nominal joint shear stress is also investigated.

Joint Deformation

As mentioned in Chapter 4, the evaluation of joint shear distortion and the estimation of joint expansion could be achieved by using the transducers that were diagonally placed in the joint core. The method for calculating joint distortion and expansion from the measured diagonal displacement was also given in Chapter 4. Joint shear deformation could be taken as an index to gauge the failure of the joint core.

Figure 5-5 Joint shear distortion and expansion of Specimen AL1

When the specimen was loaded to a storey drift ratio of $\pm 0.5\%$, there was a small amount of joint shear distortion and expansion. However, after a storey drift ratio of $\pm 0.75\%$, in which the diagonal tension cracks were observed in the joint region, the joint shear distortion increased rapidly to approximately 0.4% and the joint expansion also began to increase at this stage. This could be expected because of the relatively large tension strains measured in both the beam and column bars in the joint core. Subsequent loading cycles resulted in a sustaining increase in the joint shear distortion and joint expansion with the maximum joint shear distortion of 1.35% and the maximum joint expansion of 4.95 mm

being obtained at a storey drift ratio of 3.0%. **Figure 5-5** illustrates the sustaining increase in the joint shear distortion and expansion as the test proceeds. In summary, for the specimens without joint transverse reinforcement, the joint shear distortion and expansion increased rapidly after the diagonal tension cracking occurred in the joint core, which indicated joint shear failure.

Joint Shear Stress

The maximum nominal horizontal shear stress in the joint core of Specimen AL1 was obtained at a storey drift ratio between 1.5% and 2.0%, at which moment the maximum capacity was obtained. At these stages, strains in both the top and bottom beam longitudinal bars exceeded the yield value. The maximum nominal horizontal shear stress was calculated based on the measured strains in the beam bars with that the maximum value as 7.28 MPa, which equaled to $0.23f'_c$ or $1.29\sqrt{f'_c}$ exceeding the design limits of $0.2f'_c$ required by NZS 3101 [C1] and $1.25\sqrt{f'_c}$ required by ACI 318 [A2].

5.2.1.4 Decomposition of Horizontal Displacement

The horizontal displacement of the test specimen was composed of various components including flexural deformation, shear deformation, fixed-end rotation and joint deformation due to the joint shear distortion. In this section, these components are evaluated and presented as a percentage of the applied lateral displacement.

Figure 5-6 illustrates the components of horizontal displacement at the peak of every loading run. During the loading cycles in the elastic range, the main sources of the horizontal displacement were the unexpected and unmeasured components, which might be due to the rigid body movement. The column and beam flexural deformation plus the column and beam fixed-end rotation contributed to the remaining displacement. Among of these, the contribution of the column fixed-end rotation varied from 12.4% to 16.7%. The beam fixed-end rotation and the column flexural deformation made up a similar magnitude

to the total horizontal displacement. The contribution due to the beam flexural deformation was very limited with being only 4% up to a storey drift ratio of 0.75%. During these stages, the component caused by the joint shear distortion was not identified.

Figure 5-6 Percentage of deformation to the displacement of Specimen AL1

The contribution due to the column fixed-end rotation increased significantly in the inelastic loading cycles, which gradually increased from 28.9% to 50%. The increase of the components due to the column flexural deformation and beam fixed-end rotation was relatively small and remained nearly constantly during these stages. The maximum percentages of the column flexural deformation and beam fixed-end rotation were 15.6% and 22.6%, respectively and until the final stage, the contribution of the beam flexural deformation did not increase evidently with the maximum value reaching only 6.54%.

When the test proceeded into a storey drift ratio of 1.0%, the component of the joint shear distortion appeared and, at the first cycle of a 1.0% storey drift ratio, reached its peak value, which equaled to 10.4% of the total horizontal displacement. However, in the remaining

loading cycles, the component decreased progressively. This trend was sustained until the end of the test, where the contribution of 4.0% could almost be ignored.

5.2.1.5 Evaluation of Strains in Beam and Column Longitudinal Bars

Strains in the beam as well as in the column longitudinal bars were measured using the strain gauges. **Figure 5-7** shows the layout of these strain gauges for Specimen AL1. In the following sections, the evaluation of strains in the beam longitudinal bars is firstly conducted and followed by the evaluation of strains in the column longitudinal bars.

Figure 5-7 Layout of strain gauge positions of Specimens AL1 and AS1

Strains in Beam Longitudinal Bars

The strain profiles of the beam longitudinal bars are shown in **Figure 5-8**. Solid lines represent the strain distributions when the specimen was loaded in the positive bending, whereas dash lines are the negative cases. A progressive increasing in tension strains along the beam bars was observed. Tension strains were measured in both the beam top and bottom longitudinal bars over the whole column depth as well as a part of beam flexural compression region, which closed to the column face. It indicated that the designated “compression” bars in the beam zone alongside the joint were actually in tension. It was

evident that the beam longitudinal bars of Specimen AL1 were not well anchored in the joint core, causing an increase in the flexibility of the specimen.

At the loading cycle to a storey drift ratio of 1.0% and the second cycle to a storey drift ratio of 2.0%, the beam top and bottom bars reached the yield strain, respectively. However, in the negative loading cycle, the measured strains in the beam bottom bars did not reach the yield strain. Another feature of the strain distribution along the beam bars was that, when the yield strain was reached, the yielding penetration in the joint core was observed associated with the extensive diagonal tension cracking occurred in the joint core, which indicated that the bond slip had occurred between the beam longitudinal bars and surrounding concrete.

Strains in Column Longitudinal Bars

Figure 5-9 illustrates the strain profiles of the column longitudinal bars when the specimen was loaded in the negative direction. In the reversed loading cycles, strains in the column longitudinal bars demonstrated a symmetrical manner. In both loading directions, the column bars were extensively tensioned at a storey drift ratio of 0.75%. After that strains in the column bars increased progressively with the applied loading and reached the yield strain at a storey drift ratio of 2.0%. From the beam top face to bottom face, the column bars experienced tension stresses, which indicated that even though the ratio of the beam depth to column bar diameter was satisfied with the limit required by ACI 318 [A2], the bond slip could not be prevented in the column longitudinal bars.

(a) Beam top bar

(b) Beam bottom bar

Figure 5-8 Strain profiles of beam longitudinal bars of Specimen AL1

Figure 5-9 Strain profiles of column longitudinal bar of Specimen AL1

5.2.2 Specimens AS1 and MAS1

Specimen AS1 was a counterpart of Specimen AL1, therefore the geometries and reinforcing details were identical to those of Specimen AL1 with an exception that there was a transverse beam and a slab framed into the connection. The typical features of Specimen AS1 are summarized as follows:

- (1) Column longitudinal bars were just lap spliced above the floor level.
- (2) Column ends as well as beam ends were lightly reinforced.
- (3) No transverse reinforcement was provided within the joint core.
- (4) A transverse beam and a slab were framed into the connection.

Specimen MAS1 was taken as the comparison of Specimen AS1 with some modifications. The configuration of Specimen MAS1 was based on Specimen AS1, but the additional transverse reinforcement was provided in the beam as well as in the column potential hinge ranges. In the joint core, joint transverse reinforcement was provided to enhance the joint

shear resistance and provide the confinement to the joint panel. The typical features of this specimen can be summarized as follows:

- (1) Column longitudinal bars were just lap spliced above the floor level.
- (2) Transverse reinforcement was provided within the joint core.
- (3) A transverse beam and a slab were framed into the connection.
- (4) The spacing of transverse reinforcement within the column and beam was reduced comparing with Specimen AS1.
- (5) The end of each hoop was bent at 135 angles rather than 90 angles used in Specimen AS1.

As mentioned in Chapter 4, if the slab effect was ignored, the ratio of the column to beam flexural strength of these two specimens was 0.68, which was identical to that of Specimen AL1, and the corresponding theoretical capacity was 63.1 kN. However, if slab bars were considered fully effective in resisting a negative bending moment, this ratio would become 0.58 resulting in an enhancement of the strong beam-weak column configuration, and the theoretical capacity was 97.3 kN in either loading direction. Since the longitudinal bars reinforced in the columns and beams were not changed in these two specimens, therefore, the ratio of the column depth to beam bar diameter and the ratio of the beam depth to column bar diameter of these two specimens were both 10/1 and 20/1, respectively,

5.2.2.1 General Observation

The progression of damage to Specimens AS1 and MAS1 is comparable. **Figure 5-10** illustrates the cracking patterns of Specimen AS1 at different storey drift levels. In the elastic loading cycles, fine flexural cracks initiated and spread along the beam and column, however, only limited cracks developed in each beam and mainly concentrated along the beam bottom chord. The flexural cracking along the beam top chord was restricted at this stage, which could be attributed to the existence of the slab. In the inelastic loading cycles,

the beam flexural cracks spread along the beam's span and propagated towards the beam compression zone. Cracks along the beam top chord were detected until the loading ran into a storey drift ratio of 1.0%.

Figure 5-10 Observed cracking of Specimen AS1 at each selected storey drift level

(a) Specimen AS1

(b) Specimen MAS1

Figure 5-11 Photographs of Specimens AS1 and MAS1 after testing

Up to a storey drift ratio of 0.75%, flexural cracks formed in the bottom column through the full column depth, however, no full cracks were observed in the upper column at this stage. Due to the existence of the transverse beam passing through the joint core, diagonal tension cracks within the joint core could not be observed. As the test proceeded towards a storey drift ratio of 2.0%, some diagonal cracks were found to extend into the bottom column and to connect with the flexural cracks. It was believed that the joint diagonal tension cracks initiated at previous stages and developed in the remaining cycles. In the second loading cycle to a 2.0% storey drift ratio, the concrete in the column compression zone was crushed and started to spall. Also, the cracks on the column fixed-end interface opened gradually to a width of about 4 mm on each tension side of the column. Since this loading ran, the lateral displacement started to concentrate in the column fixed-end rotation.

At later test stages, diagonal torsion cracks were observed in the transverse beam close to the joint extending at 45 angles and concentrating within a small area (see Figure 5-11). It can be concluded that the slab bars parallel to the beam longitudinal bars participated in resisting the bending moment and then transferred the inter-forces by means of the torsional moment within the transverse beam. **Figure 5-11** presents photographs of Specimens AS1 and MAS1 after the test.

5.2.2.2 Hysteretic Behaviour

Specimen AS1

Figure 5-12 illustrates the storey shear force versus horizontal displacement relationship for Specimen AS1. Also shown are the theoretical capacities of the test unit. Solid lines present the theoretical capacities without consideration of the presence of the slab while dash lines are the theoretical capacities assuming that all the beam longitudinal bars and slab bars yielded simultaneously. In other words, it is assumed that slab bars within the whole slab width will effectively participate in resisting the lateral loading.

The first yield of Specimen AS1 occurred at a displacement of 19.6 mm corresponding to the storey drift ratio of 0.6% in the positive loading direction. At this moment, the specimen also reached its theoretical capacity. The positive yield strength occurred earlier than negative yield strength, which occurred at a 1% storey drift ratio in the negative loading direction. The maximum capacities were obtained at storey drift ratios of 2% and 1.5%, which were 80 kN in the positive and 70 kN in the negative loading directions indicating that the specimen took over-strength factors at 27% and 11%, respectively. If compared with Specimen AL1, the maximum capacities of Specimen AS1 were 51% and 32% higher than those of Specimen AL1 in the same loading directions. It was evident that the participation of a slab enhanced the moment resisting capacity of the specimen, but the slab bars were not fully effective to resist bending moments and therefore, the specimen didn't reach the theoretical capacity with considering the slab effect in both loading directions.

Figure 5-12 Storey shear force versus horizontal displacement relationship for Specimen AS1

In the first loading cycle to a storey drift ratio of 3.0%, the recorded storey shear force dropped to the theoretical level and the hysteretic loop showed apparent pinching. The

reduction of the load-carrying capacity and the degradation of the stiffness were mainly caused by the hysteretic loop pinching induced from concrete cracking. The stiffness of the specimen continuously degraded with the residual load-carrying capacity being about 53 kN in the final loading stage. When the specimen was loaded further to a displacement of 109 mm and the test was terminated at this stage because the LVDT monitoring the top displacement ran out of travel-distance.

Specimen MAS1

Figure 5-13 illustrates the storey shear force versus horizontal displacement relationship for Specimen MAS1. The hysteretic behaviour of Specimen MAS1 was similar to that of Specimen AS1. In the positive loading direction, the first yield occurred at a displacement of 23.8 mm corresponding to the storey drift ratio of 0.8%. At this moment, the specimen also reached its theoretical capacity. On the other hand, the theoretical capacity was reached in the negative loading direction at a storey drift ratio of 1.0%. The maximum capacities were obtained at storey drift ratios of 1.5% and 2.0%, which were 73.8 kN in the positive and 69.6 kN in the negative loading directions indicating that the specimen took over-strength factors at 17% and 10%, respectively. If compared with Specimen AS1, the maximum capacity of Specimen MAS1 was 92% of that of Specimen AS1 in the positive loading direction. But, the maximum capacities of these two specimens in the negative loading direction were almost same indicating that the modification of reinforcing details did not improve the overall performance significantly.

Significant pinching and stiffness degradation were observed after the second cycle of a storey drift ratio of 1.5% and strength degradation was not evident until a storey drift ratio of 3.0% was reached. During the test, the upper bound of the theoretical capacity couldn't be reached in this unit. The specimen was finally loaded further to a displacement of 109 mm or a drift ratio of 3.8%. The residual load-carrying capacity was about 55 kN after the final loading was applied.

Figure 5-13 Storey shear force versus horizontal displacement relationship for Specimen MAS1

5.2.2.3 Joint Behaviour

Due to the presence of transverse beams, the joint shear distortion and expansion of Specimens AS1 and MAS1 could not be experimentally evaluated using the aforementioned method for evaluation of joint shear deformation.

The maximum nominal joint shear stress of Specimens AS1 and MAS1 were 8.49 MPa and 7.98 MPa or equaled to $0.27 f'_c$ or $1.52 \sqrt{f'_c}$ and $0.25 f'_c$ or $1.43 \sqrt{f'_c}$, respectively. All these values exceeded the limits either required by NZS 3101 [C1] or ACI 318 [A2].

5.2.2.4 Decomposition of Horizontal Displacement

Specimen AS1

Figure 5-14 illustrates the components of horizontal displacement of Specimen AS1 at the peak of every loading run. In the beginning of the test, some unexpected and unmeasured components, which might include the joint shear deformation contribution, were observed.

Although the contributions of these components to the total horizontal displacement were much larger in the percentage at the early stages, their magnitude of displacement was small and remained constantly during the test and hence their influences were limited.

Figure 5-14 Percentage of deformation to the displacement of Specimen AS1

The column and beam fixed-end rotation plus column flexural deformation contributed to the remaining of displacement. The contribution due to the column fixed-end rotation was very important as it gradually increased from 23.6% to 50.9% in the elastic range. In the following inelastic range, however, its contribution was unchanged in percentage terms. The increase of components due to the column flexural deformation and beam fixed-end rotation was relatively small during these loading stages with the maximum percentages of the column flexural deformation and beam fixed-end rotation being about 23.8% and 30.4%, respectively. Until the final stage of the test, the evident increase in beam flexural deformation was limited with the contribution no more than 7%.

Specimen MAS1

Figure 5-15 illustrates the components of horizontal displacement of Specimen MAS1 at the peak of every loading run. Some unexpected and unmeasured components, which appeared mainly due to the joint shear deformation contribution, were observed. Compared with Specimen AS1, the contributions of these components to the total horizontal displacement were slightly higher; therefore it was likely that the slight confinement could not improve the joint performance significantly.

The contributions due to the beam flexural deformation and fixed-end rotation were very limited and therefore, it can be concluded that the beam stiffness was enhanced through the slab participation as well as the improved beam confinement.

Figure 5-15 Percentage of deformation to the displacement of Specimen MAS1

With reference to **Figure 5-15**, it can be seen that the main sources of the horizontal displacement were the column fixed-end rotation and flexural deformation with these two components increasing gradually during the test. The component due to the column flexural deformation increased from a percentage of 0.33% to 25.4% while the increase of

the component due to the column fixed-end rotation was relatively higher at about 2.37% to 45.5%.

5.2.2.5 Evaluation of Strains in Beam and Column Longitudinal Bars

Since the reinforcing details of Specimen AS1 were identical to those of Specimen AL1, the strain gauge positions of Specimen AS1 can be referred to **Figure 5-7**. The distributions of strain gauges of Specimen MAS1 are shown in **Figure 5-16**. Comparing with Specimens AL1 and AS1, more strain gauges were provided in Specimen MAS1 to measure the strain development during the test.

Figure 5-16 Layout of strain gauge positions of Specimen MAS1

Strains in Beam Longitudinal Bars

Specimen AS1

The strain profiles of the beam longitudinal bars of Specimen AS1 are shown in **Figure 5-17**. Up to the final stage, a gradual increase in tension strains along the beam bars was observed. Tension strains were measured in both the beam top and bottom longitudinal bars over the whole column depth as well as a part of the beam flexural compression region, which closed to the column face. At a storey drift ratio of 2.0%, the beam top bars were

yielded in both the positive and negative loading cycles. For the beam bottom bars, the yield strain was reached at a storey drift ratio of 3.0%; however, it was only measured in the right hand side.

Specimen MAS1

In Specimen MAS1, strains in the beam longitudinal bars were similar to those of Specimen AS1. At a storey drift ratio of 2.0%, the beam top bars were yielded in the negative loading direction, but strains in the beam bottom bars were not measured completely since some strain gauges were damaged during the construction. From the readings of the remainder strain gauges, it was found that tension strains in the beam bottom bars did not reach the yield strain indicating that the beam bottom bars retained their behaviour in the elastic range during the test.

Strains in Column Longitudinal Bars

Figures 5-18 and **5-19** show the strain profiles of the column longitudinal bars of Specimens AS1 and MAS1, respectively. As the test ran towards a storey drift ratio of 2.0%, the column bars in the tension side reached the yield strain. From the beam top face to bottom face, the column bars experienced tension stresses. Yielding penetration could not be prevented in these specimens indicating that the column longitudinal bars were not well anchored in the joint core. It was found that the magnitude of strains after yielding was much higher in Specimen AS1 than that in Specimen MAS1.

Figure 5-17 Strain profiles of beam longitudinal bars of Specimen AS1

Figure 5-18 Strain profiles of column longitudinal bar of Specimen AS1

Figure 5-19 Strain profiles of column longitudinal bar of Specimen MAS1

5.3 Test Results of Specimens AL2 and AS2

5.3.1 Specimen AL2

The geometries and reinforcing details of Specimen AL2 were identical to those of Specimen AL1 but the column of Specimen AL2 could be bent about its strong axis. The typical features of this specimen are summarized as follows:

- (1) Column longitudinal bars were just lap spliced above the floor level;
- (2) Column ends as well as beam ends were lightly reinforced;
- (3) No transverse reinforcement was provided within the joint core.

The ratio of the column to beam flexural strength of the specimen was 1.26, and thus plastic hinges were expected to form in the beams and lead to a strong column-weak beam structural configuration. Due to different orientation, the ratio of the column depth to beam bar diameter was $h_c/d_b = 20/1$ and the ratio of the beam depth to column bar diameter was still $h_b/d_c = 20/1$. The theoretical capacity of the specimen was 63.9 kN, which was slightly higher than that of Specimen AL1 in either loading direction. Following sections report the test results of this specimen.

5.3.1.1 General Observation

Cracking patterns were observed at different storey drift levels as shown in **Figure 5-20**. When the loading ran from the first cycle of a 0.5% storey drift ratio into the reversed cycle, flexural cracks appeared in the beam and limited fine cracks were found in the joint region as well. In the subsequent cycles to a storey drift ratio of 0.75%, the flexural cracks spread along the beam span on both the top and bottom chords and propagated vertically; as a result, full depth cracks were developed.

In the joint region, smallish diagonal tension cracks were developed and concentrated in a corner, while the flexural and shear cracks were also generated in the column. During the subsequent cycles, the amount of column cracks did not increase, which indicated that the column flexural deformation contribution to the horizontal displacement was not important and the column retained its behaviour in the elastic range.

Figure 5-20 Observed cracking of Specimen AL2 at each selected storey drift level

Figure 5-21 Photograph of Specimen AL2 after testing

In the inelastic cycles to a storey drift ratio of 2.0%, the diagonal tension cracks increased intensively within the joint core and main diagonal concrete compression struts formed due to the generation of corner-to-corner diagonal cracks. The width of these diagonal cracks

was measured from 0.4 mm to 2 mm. At these stages, the concrete was crushed in the beam flexural compression zone. **Figure 5-21** presents a photograph of Specimen AL2 after the test.

5.3.1.2 Hysteretic Behaviour

Figure 5-22 illustrates the storey shear force versus horizontal displacement relationship for Specimen AL2. The obtained yield displacement was 21 mm corresponding to the storey drift ratio of 0.8%. This was about three times that of the predicted yield displacement of 6.75 mm. At this moment, the specimen reached its theoretical capacity of 63.9 in either loading direction. The maximum capacities were obtained at a storey drift ratio of 1.5%, which were 82 kN in the positive and 75 kN in the negative loading directions indicating that the specimen took over-strength factors at 28% and 17%, respectively.

Figure 5-22 Storey shear force versus horizontal displacement relationship for Specimen AL2

From the drift ratio of 1.5%, significant pinching was observed in the hysteretic loops and, in the repeated cycle, the specimen started to lose its load-carrying capacity at 11.9% and subsequently, strength and stiffness degradation was evidently. At the second cycle of the loading run to a drift ratio of 3.0%, the specimen lost its load-carrying capacity at 20% indicating that the specimen failed at this stage. At the first semi-cycle of a 3.8% storey drift ratio, the load-carrying capacity suddenly dropped to 42 kN and so for the reasons of safety, the test was withheld at this moment.

5.3.1.3 Joint Behaviour

Joint Deformation

The measured joint shear distortion and expansion is shown in **Figure 5-23**. During the test, the joint shear distortion and expansion progressively increased with the loading run. In the loading to a $\pm 0.5\%$ storey drift ratio, there was a little joint deformation, but after the loading went to a $\pm 0.75\%$ storey drift ratio, in which the diagonal tension cracks were observed in the joint, the joint shear distortion increased rapidly to 0.65%, while the joint expansion also began to increase. Subsequent loading cycles resulted in a consistent increase in the joint expansion as shown in **Figure 5-23**. The joint shear distortion also increased consistently with the maximum joint shear distortion of 1.17% and the maximum joint expansion of 7.5 mm being obtained at a storey drift ratio of 3.0%.

Joint Shear Stress

The maximum nominal horizontal shear stress of Specimen AL2 was 6.26 MPa, or equalled to $0.2 f'_c$ or $1.06 \sqrt{f'_c}$, which satisfied with the design limits either required by NZS 3101 [C1] or ACI 318 [A2].

Figure 5-23 Joint shear distortion and expansion of Specimen AL2

5.3.1.4 Decomposition of Horizontal Displacement

Figure 5-24 illustrates the components of horizontal displacement at the peak of every loading run. During the loading cycles in the elastic range, some unexpected and unmeasured components were also found. Among these components, the column and beam flexural deformation was not the important factor to the horizontal displacement. The contributions of these two factors were very limited and almost unchanged with the maximum percentages being about 6.1% and 11.2%, respectively, until the end of the test.

Resources contributing to the remainder of the displacement were the column fixed-end rotation, beam fixed-end rotation and joint shear distortion. Among of these, the contribution due to the column fixed-end rotation increased progressively and varied from 3.4% to 31.6%, whereas the contribution due to the beam fixed-end rotation did not change evidently during the early stages, but it increased gradually when the test proceeded into a storey drift ratio of 2.0% and reached its peak value at a percentage of 39.2% in the final stage. The maximum component due to the joint shear distortion was apparent at the point

when the loading was applied to a storey drift ratio of 3.0% and contributed to the lateral displacement at a percentage of 46.3%.

Figure 5-24 Percentage of deformation to the displacement of Specimen AL2

5.3.1.5 Evaluation of Strains in Beam and Column Longitudinal Bars

Figure 5-25 shows the layout of strain gauges on the beam and column bars of Specimen AL2. The following section reports the local behaviour by investigating the strains in the beam longitudinal bars which is then followed by an evaluation of the strains in the column longitudinal bars.

Figure 5-25 Layout of strain gauge positions of Specimens AL2 and AS2

Strains in Beam Longitudinal Bars

The strain profiles of the beam top and bottom bars are shown in **Figure 5-26**. Up to a loading cycle of 3.0% storey drift ratio, the gradual increased strains in the beam bars were detected. The beam top bars reached the yield strain at a storey drift ratio of 3.0% while strains in the beam bottom bars did not reach the yield strain during the whole loading cycles. Tension strains were measured in the beam longitudinal bars over the whole column depth during the test. Bars in the beam compression zone only developed compression strains indicating that the beam longitudinal bars of Specimen AL2 were well anchored in the joint core.

Strains in Column Longitudinal Bars

The strain profiles of the column longitudinal bars are shown in **Figure 5-27**. In the loading cycle to a storey drift ratio of 0.75%, in which the diagonal tension cracks were observed in the joint, tension strains in the column longitudinal bars were measured through the joint. Subsequently, the tension strains in the column bars increased gradually as the test proceeded, however, they did not reach the yield strain. It could be concluded that the

column longitudinal bars of Specimen AL2 remained in elastic range up to the end of the test.

(a) Beam top bar

(b) Beam bottom bar

Figure 5-26 Strain profiles of beam longitudinal bars of Specimen AL2

Figure 5-27 Strain profiles of column longitudinal bar of Specimen AL2

5.3.2 Specimen AS2

Specimen AS2 was a counterpart of Specimen AL2, therefore the geometries and reinforcing details were identical to those of Specimen AL2 with the exception that there was a transverse beam and a slab framed into the connection. The typical features of Specimen AS2 are summarized as follows:

- (1) Column longitudinal bars were just lap spliced above the floor level.
- (2) Column ends as well as beam ends were lightly reinforced.
- (3) No transverse reinforcement was provided within the joint core.
- (4) A transverse beam and a slab were framed into the connection.

The column to beam flexural strength ratio without the slab effect, which was 1.26, was identical to that of Specimen AL2. However, this ratio would become 1.09 with considering of the slab effect. Since the reinforcing details were not changed in the specimen, therefore, the ratio of the column depth to beam bar diameter and the ratio of the

beam depth to column bar diameter were identical with those of specimen AL2. As mentioned in Chapter 4, if slab effect was ignored when calculating load-carrying capacity, Specimen AS2 had an identical theoretical capacity of 66.7 kN, whereas if slab bars were considered fully effective in resisting a negative bending moment, the theoretical capacity was 102.7 kN in either loading direction. The following sections report the test results of the specimen.

5.3.2.1 General Observation

Figures 5-28 illustrates the cracking patterns of Specimen AS2 at different storey drift levels. In the loading cycle to a storey drift ratio of 0.5%, flexural cracks initiated in the beam and concentrated in the beam bottom chord. Up to a storey drift ratio of 0.75%, limited shear cracks were observed in the top chord of the beam just below the bottom face of slab. These cracks spread along the beam span and propagated vertically corresponding to the increased loading. The bending in the main beam resulted in both tension and shear forces in the plane of the slab. Therefore, the shear force, bending and torsion moments were induced in the transverse beam and as a result, minor torsion and shear cracks in the transverse beam of Specimen AS2 were found close to the joint. **Figure 5-29** presents photographs of Specimen AS2 after the test.

Different cracking patterns were detected in the column. During the elastic range, flexural cracks were observed in the lower column, which were fully developed through the column depth in the loading cycle to a storey drift ratio of 1.0%, whereas the cracks were evident in the upper column until the test ran into the inelastic range when the cracking was mainly due to the column shear.

In the joint core, diagonal tension cracks were found in the early stages. However, due to the presence of the transverse beam, cracks in the joint core could only be detected outside of the confined region (see **Figure 5-29(a)**). In the subsequent loading cycles, cracks

generated intensively and therefore, it might be expected that the diagonal concrete compression strut failure dominated the failure mechanism of the joint.

Figure 5-28 Observed cracking of Specimen AS2 at each selected storey drift level

(a) Cracks in the beam, column and joint

(b) Cracks in the joint region and transverse beam

Figure 5-29 Photographs of Specimen AS2 after testing

5.3.2.2 Hysteretic Behaviour

Figure 5-30 illustrates the storey shear force versus horizontal displacement relationship for Specimen AS2. The first yield displacement was 14 mm and equaled to a storey drift ratio of 0.6%. This was about two times that of the predicated yield displacement of 6.54. At this moment, the specimen reached its theoretical capacity of 66.7 kN in either loading direction. The maximum capacities were obtained at a storey drift ratio of 2.0%, which were 98 kN in the positive and 83 kN in the negative loading directions indicating that the specimen took over-strength factors at 47% and 24%, respectively. If compared with Specimen AL2, the maximum capacities of Specimen AS2 were 19% and 11% higher than those of Specimen AL2 in the same loading directions.

Subsequently, the specimen lost its load-carrying capacity progressively until the theoretical value was reached. Limited pinching was observed after the second cycle of a storey drift ratio of 1.5%. After the second cycle of a storey drift ratio of 2.0% up until the end of the test, pinching and stiffness degradation were obvious.

Figure 5-30 Storey shear force versus horizontal displacement relationship for Specimen AS2

5.3.2.3 Joint Behaviour

The maximum nominal horizontal shear stress in the joint core of Specimen AS2 was 6.34 MPa or equaled to $0.2 f'_c$ or $1.06 \sqrt{f'_c}$, which satisfied the design limits either specified in NZS 3101 [C1] or ACI 318 [A2].

5.3.2.4 Decomposition of Horizontal Displacement

Figure 5-31 illustrates the components of horizontal displacement at the peak of every loading run. In the beginning of the test, some unexpected and unmeasured components, which were mainly due to the joint deformation, were contributing significantly to the total horizontal displacement with the maximum percentage of 44.5% being reached at a storey drift ratio of 0.5% in the elastic range.

Figure 5-31 Percentage of deformation to the displacement of Specimen AS2

The contribution from the column flexural deformation was not very significant during the test. This coincided with the description mentioned above that the column deformed mainly due to the “column shear” rather than “column flexure”. Other resources contributing to the

remainder displacement were the column fixed-end rotation plus the beam fixed-end rotation and beam flexural deformation. Amongst these, the contributions of the column and beam fixed-end rotation varied from 20% to 43% with the increase associated with the overall deformation demand. The contribution due to the beam flexural deformation increased gradually until a 2.0% storey drift ratio with the magnitude of this contribution varying from 10% to 15%.

5.3.2.5 Evaluation of Strains in Beam and Column Longitudinal Bars

Strains in Beam Longitudinal Bars

The layout of strain gauges on the beam and column bars of Specimen AS2 is shown in **Figure 5-25**. Up to the end of the test, tension strains along the beam longitudinal bars were observed within the joint core. At a storey drift ratio of 2.0%, the beam top bars reached the yield strain in either loading direction. Compared with the strain profiles of Specimen AL2, tension strains were found not only in the beam bars within the joint core but also in the compression zone of the beam. Strains in the beam bottom bars were not measured since some strain gauges were damaged during the construction. **Figure 5-32** illustrates the strain profiles of the beam top bars of Specimen AS2.

Strains in Column Longitudinal Bars

The strain profiles of the column longitudinal bars are shown in **Figure 5-33**. Even though the strains in the column longitudinal bars increased gradually during the test, however, they did not attain the yield strain until a storey drift ratio of 3.0% was reached. From the beam top face to bottom face, the column bars experienced tension stresses which indicated that the column longitudinal bars were not well anchored in the joint core.

Figure 5-32 Strain profiles of beam top bar of Specimen AS2

Figure 5-33 Strain profiles of column longitudinal bar of Specimen AS2

5.4 Finite Element Model Development

In this section, the behaviour of the tested specimens was investigated using 3D finite element analyses. The method for developing of 3D nonlinear finite element models is firstly introduced followed by the comparison of the analytical results with the experimental results. The general information with regards to finite element modeling can be found in Chapter 3.

5.4.1 Material Properties

In 3D finite element analyses, the structural response is sensitive to the selected constitutive relationship. Natio [N3] carried out a series of analyses adopting different modeling criteria for evaluating 3D finite element models. It was found that the most accurate estimation of a response was achieved by using the total strain model. In addition, a rotated-crack model provided a good estimate of responses as well as more rational formulation. These summaries were in accordance with the recommendations by the DIANA [D3] manuals. Therefore, in the current study, the technique used for creating constitutive law is still based on the total strain formulation, where the constitutive relationship is defined in the principal strain state. **Table 5-1** shows the material parameters used for finite element analysis.

5.4.2 Modeling of the Interaction between Steel and Concrete

In the finite element analyses, the numerical models of the corresponding test specimens were subjected to the reversed cyclic loadings like those applied on the test specimens. It is believed that after several cycles of loading, the loss of rigidity of the structure was partly due to the slippage of the reinforcement and the steel-concrete bond completely reduced to a simple friction [O1]. To model the bond-slip behaviour between the steel and surrounding concrete, a nonlinear force-deflection element was used. **Figure 5-34(a)** shows an interface element between a 4-nodes element and a truss element, which models the slippage between the bars and surrounding concrete.

The fundamental of bond-slip model suitable for the bars anchored in the joints was established by Eligehausen *et al.* [E3] as shown in **Figure 5-34**. In DIANA [D3], a bond-slip element is classified as an interface element, which describes the interface behaviour in terms of a relationship between the normal and shear traction as well as a relationship between the normal and shear relative displacement across the interface. The relationship between the normal traction and normal relative displacement is assumed to be linear elastic, whereas the relationship between the shear traction and slip is assumed as a nonlinear function. The shear relations for the positive and negative values of slippage are assumed to be equal. The unloading and reloading relationship is modeled using a secant approach. Once the slippage reversed, a straight line back to the origin is followed. Beyond the origin, the bond-slip curve (with an opposite sign) is inserted. DIANA [D3] offers two predefined curves for the relationship between the shear traction and slippage: a Cubic function according to Dörr [D4] model and a Power Law relationship proposed by Noakowski [N2]. In this thesis, the power law was used in the all numerical models.

Table 5-1 Parameters of constitutive models

Material Parameters		Values	Units
Concrete tension strength	f_t'	1.6	MPa
Concrete compression strength	f_c'	32.8	MPa
Concrete elastic modulus	E_c	20000	MPa
Concrete poisson ratio	ν	0.16	
Steel tension yield strength	f_y	473	MPa
Steel ultimate strain	ε	0.16	
Steel elastic modulus	E_s	2.1e+5	MPa
Fracture energy	G_f	0.2

Figure 5-34 Interface elements and constitutive law of bond slip relationship

5.4.3 Mesh Generation

The analytical models were set up to match the gross dimensions and reinforcing details of the as-built subassemblies. Among the five numerical models, Models AL1 and AL2 were discretized in two dimensions to provide more accurate results and the others were modelled in three dimensions. In three dimensional models, the concrete was discretized using an 8-nodes brick element. A $2 \times 2 \times 2$ Gauss integration scheme, which is recommended by the DIANA [D3] manual, was adopted. Over the entire system, the reinforcement was modeled as a discrete truss element attached to the edges of the concrete elements. **Figures 5-35 and 5-36** show the mesh of the finite element models.

(a) Model AL1

(b) Model AL2

Figure 5-35 Two-dimensional finite element models AL1 and AL2

(a) Model AS1

(b) Model AS2

Figure 5-36 Three-dimensional finite element models AS1 and AS2

5.4.4 Boundary Condition

In the all finite element models, nodes on the symmetrical axis along the transverse direction on the column bottom face were fixed in three directions to simulate the pin supports at the column ends as such the models were allowed to rotate in the X-Y plane. In the test, the beam ends were constrained using steel legs which were pinned at floor. Corresponding to this boundary condition, in the finite element models, the beam ends were constrained using pin-roller supports. Therefore, at the beam ends, the displacements along the vertical and transverse directions were restrained and the horizontal direction was left to be free. To avoid stress concentration and to reduce computational time, beam and column ends were modeled as rigid bodies. **Figure 5-37** illustrates the boundary conditions of the finite element models.

Figure 5-37 Boundary conditions of finite element models

5.4.5 Applied Displacement History

A horizontal load was applied as a nodal displacement at the nodes along the centre line of the top reaction block and the displacement histories corresponded closely to the loading histories applied in the test. However, in order to reduce the time cost, only one cycle at each loading level was applied to the finite element models.

5.5 Comparison of Experimental and Analytical Results

5.5.1 Cracking Patterns

Using DIANA V7.2 [D3], cracking patterns cannot be captured graphically in three dimensional models as such the comparison of cracking patterns can only be applied for Models AL1 and AL2, which were set up in two dimensions. **Figures 5-38 and 5-39** illustrate the cracking patterns observed during the test and obtained through a finite element approach.

In the analyses, diagonal tension cracks started to occur as early as the models were loaded to a drift ratio of 0.5% and subsequently the extensive diagonal cracking was found concentrated within the joint core.

Figure 5-38 Comparison of cracking patterns of Specimen AL1 and Model AL1 (Experimental vs. Analytical)

Figure 5-39 Comparison of cracking patterns of Specimen AL2 and Model AL2 (Experimental vs. Analytical)

5.5.2 Hysteretic Behaviour

Figures 5-40 to 5-44 illustrate the comparison of hysteretic behaviour obtained through finite element analyses and experimental investigations. It can be seen that the overall behaviour between the analytical and experimental results is basically in good agreement. According to the finite element analysis results, the maximum capacities were reached around a storey drift ratio of 1.5% and after that, strength and stiffness degradation was

obvious. In the all analytical hysteretic loops, significant pinching was observed. These phenomena were similar to those observed during the test.

If compared the analytical with the experimental storey shear forces, it is found that Model AL1 showed a slightly higher load-carrying capacity at early stages. At a storey drift ratio of 1.5%, Model AL1 exceeded the theoretical capacity and gained its maximum capacity of 65 kN, which was of 22% higher than the measured value in the positive loading direction. Whereas, Specimen AL2 presented a higher load-carrying capacity comparing with Model AL2. The maximum capacity of Model AL2, which occurred at 0.5% storey drift ratio in either loading direction, was 90% of that of Specimen AL2. Model MAS1 showed a higher load-carrying capacity comparing with Specimen MAS1. The maximum capacity of Model MAS1 was 11% and 5% higher than the measured values in the positive and negative loading direction, respectively. However, the differences in the analytical and experimental results were acceptable and therefore, the analytical models could be utilized to predict the other behaviour.

Figure 5-40 Comparison of hysteretic behaviour between Specimen AL1 and Model AL1

Figure 5-41 Comparison of hysteretic behaviour between Specimen AS1 and Model AS1

Figure 5-42 Comparison of hysteretic behaviour between Specimen MAS1 and Model MAS1

Figure 5-43 Comparison of hysteretic behaviour between Specimen AL2 and Model AL2

Figure 5-44 Comparison of hysteretic behaviour between Specimen AS2 and Model AS2

5.5.3 Evaluation of Strains in Beam and Column Longitudinal Bars

5.5.3.1 Strains in Beam Longitudinal Bars

Figures 5-45 to 5-48 show the comparison of strain profiles of the beam longitudinal bars. The compared results presented here only consist of the strain profiles of the beam top bars when the models were loaded in a single loading direction.

Note that the limited measuring data in the experimental specimens resulted in inaccuracies in the strain distributions. Before a storey drift ratio of 2.0% was reached, the strain magnitude of the analytical and experimental models was in good agreement, however, after the yield strain was reached, the strain magnitude of the analytical models was much higher than that obtained through experimentations.

Figure 5-45 Comparison of strain profiles of beam longitudinal bars between Specimen AL1 and Model AL1

Figure 5-46 Comparison of strain profiles of beam longitudinal bars between Specimen AS1 and Model AS1

Figure 5-47 Comparison of strain profiles of beam longitudinal bars between Specimen AL2 and Model AL2

Figure 5-48 Comparison of strain profiles of beam longitudinal bars between Specimen AS2 and Model AS2

5.5.3.2 Strains in Column Longitudinal Bars

Figures 5-49 to 5-52 show the comparison of strain profiles of column longitudinal bars. In general, the strain distribution as well as strain magnitude of the analytical and experiential models was basically in good agreement with only an exception being observed in Model AL1, where the yield strain was not reached in the analysis.

5.5.4 Joint Shear Stress

Table 5-2 tabulates the comparison of calculated joint shear stress through experimental and analytical approaches.

It should be noted that, when calculate the nominal horizontal joint shear stress in the analytical models, the tension forces induced by slab bars were not taken into account because it was difficult to estimate how much they could be transferred into the joint. If assuming the tension forces of slab bars were fully transferred into the joint core, the joint shear stresses of Models AS1, MAS1, and AS2 were re-calculated and showed as below:

Model AS1: $v_{jk} = 9.3$ (MPa) equals to $0.29 f'_c$ or $1.66 \sqrt{f'_c}$

Model MAS1: $v_{jk} = 9.4$ (MPa) equals to $0.30 f'_c$ or $1.68 \sqrt{f'_c}$

Model AS2: $v_{jk} = 6.75$ (MPa) equals to $0.21 f'_c$ or $1.13 \sqrt{f'_c}$

If comparing the analytical and experimental joint shear stresses in **Table 5-2**, it is clear that, when ignoring the slab participation, Models AS1, MAS1 and AS2 showed lower joint shear stress demands than those observed in the experiment. In the test specimens, the extra joint shear forces might attribute to the tension forces induced by slab bars. However, when comparing the experimental results with the re-calculated analytical results, it is understood that the slab bars did not fully effective to participate in transfer the forces.

Table 5-2 Nominal joint shear stress in analytical and experimental models

Model	Joint shear stress (Experimental)			Joint shear stress (Analytical)		
	(MPa)	f'_c (MPa)	$\sqrt{f'_c}$ (MPa)	(MPa)	f'_c (MPa)	$\sqrt{f'_c}$ (MPa)
AL1	7.28	0.23	1.29	7.3	0.23	1.30
AS1	8.5	0.27	1.51	6.86	0.22	1.22
MAS1	8.0	0.25	1.42	6.96	0.22	1.25
AL2	6.26	0.2	1.06	5.1	0.16	0.87
AS2	6.34	0.2	1.06	4.92	0.16	0.82

5.5.5 Evaluation of Effective Slab Width

In an effort to estimate the effective slab width, the strain distribution in the slab bars is evaluated in this section. **Figures 5-53** and **5-54** show the strain distributions in the beam longitudinal bars and slab bars which along the transverse direction at the column faces.

It can be seen from **Figure 5-53** that the beam bars reached the yield strain at a storey drift ratio of 2.0%. However, the magnitude of strains in the slab bars was much lower until the final stage of analysis. This can be explained that, due to the weak column-strong beam configuration, the deformation demand on the beam element is limited and the presence of

the slab enhanced the strong beam behaviour resulting in a further reduction of beam deformation. In addition, the transverse beam, which was lightly reinforced, could lose their torsional moment bearing capacity due to the torsional failure in an early stage, and as a result, the tension forces in the slab bars could not be developed and be transferred into the joint core.

Model AS2 shows a different strain distribution. Within a distance of 200 mm on each side of the beam face, the slab bars associated with the beam bars yielded at a storey drift ratio of 1.0%, while the total effective slab width could be taken as the beam width plus the beam depth on the each side of the beam. It is evident that the deformation demand on the beam element in this specimen was high and as a result, the slab bars in the effective width were significantly stressed in tension.

Figure 5-49 Comparison of strain profiles of column longitudinal bars between Specimen AL1 and Model AL1

Figure 5-50 Comparison of strain profiles of column longitudinal bars between Specimen AS1 and Model AS1

Figure 5-51 Comparison of strain profiles of column longitudinal bars between Specimen AL2 and Model AL2

Figure 5-52 Comparison of strain profiles of column longitudinal bars between Specimen AS2 and Model AS2

Figure 5-53 Strain distributions in the beam top bars and slab bars of Model AS1

Figure 5-54 Strain distributions in the beam top bars and slab bars of Model AS2

5.6 Discussion of Experimental Results of Lightly Reinforced Beam-Column Joints

5.6.1 Load-Carrying Capacity and Strength Degradation

Figure 5-55 illustrates the comparison of the overall behaviour among Specimens AL1, AS1, and MAS1, which are presented using storey shear force-horizontal displacement envelope curves. These curves indicate the effect of slab participation and modification of reinforcing details on the load-carrying capacity. Specimen AL1 showed the lowest maximum capacity at 53 kN, which is 0.88 times that of the theoretical capacity. Specimen AS1 exhibited the highest maximum capacity in group one with this value being 80 kN or 1.27 times that of the theoretical capacity and Specimen MAS1 reached the maximum capacity at 73.8 kN or 1.17 times of the theoretical capacity.

The comparison of the behaviour of Specimens AL2 and AS2 in **Figure 5-56** illustrates the effect of slab participation on the load-carrying capacity of beam-column connections in the strong direction of the moment resisting frame. Specimen AL2 reached the maximum capacity of 82 kN at a storey drift ratio of 1.5%, which is 1.28 times that of the theoretical capacity and Specimen AS2 exhibited the highest load-carrying capacity of 98.5 kN, which is 1.5 times that of the theoretical capacity at a storey drift ratio of 2.0%.

The strength of the specimens with a slab was increased compared with those specimens without a slab. In group one, Specimens AS1 and MAS1 were able to carry higher loads than Specimen AL1 from the start of the test. In group two, the strength of Specimens AL2 and AS2 was quite close before a 1.0% storey drift ratio, but in the subsequent loading cycles, the strength of Specimen AS2 was higher than that of Specimen AL2.

The reduction of spacing of the transverse reinforcement in the beam and column potential hinge ranges and the application of joint transverse reinforcement in Specimen MAS1 caused a certain amount of confinement to the core concrete and as a result, the resistance to lateral expansion of the concrete and the increase in strength and ductility could be expected. However, in this specimen, the improvement of the overall response due to the modification of reinforcing details was limited. It can be found from **Figure 5-55** that the strength of Specimen MAS1 was higher than that of Specimen AS1 only when the deformation level extended to a 1.5% storey drift ratio and after that the strength difference between these two specimens was not significant.

A comparison of the behaviour of the specimens in **Figures 5-57** and **5-58** illustrates the effect of flexural strength ratio on the overall response of beam-column connections. In Specimens AL1 and AS1, the behaviour of the specimens was governed by the column behaviour and therefore, the strength of these specimens was inferior to their counterparts.

Figures 5-55 to 5-58 illustrate that the response of the specimens was significantly affected by the cyclic loadings. Due to the crack formation, bond-slip, or a combination of these failure mechanisms, the strength degradation of the specimens was more noticeable as the test proceeded.

The difference in load-carrying capacities between the subsequence loading cycles was much larger particularly in the inelastic range. The strength degradation varied from 11% to 21% and the tendency was constantly increased with the applied loading. Comparing the results of Specimens AL1 and AL2 with their counterparts at the same demand levels, it is noted that the percentage of decrease in the load-carrying capacity of these specimens was higher than that of the specimens with a slab. Therefore, the effect of the failure mechanism on the overall strength degradation of the specimens was reduced due to the slab participation.

5.6.2 Comparison of Stiffness Degradation

In order to calculate the theoretical stiffness, it is assumed that the effective moment of inertia of the beams and columns was $0.5I_g$ and that the deformation due to the joint shear distortion contributed to the lateral displacement by 20%, where I_g is the moment of inertia based on the un-cracked gross concrete area. Therefore, the theoretical initial stiffness, K_{th} , is given in **Table 5-3** with the measured initial stiffness being also shown in this table.

Table 5-3 Theoretical and initial stiffness of lightly reinforced test specimens

Specimen	K_{th} (kN/mm)	$K_{initial}$ (kN/mm)	Stiffness ratio
AL1	4.8	2.1	43.8%
AS1	5.0	3.1	62.0%
MAS1	5.0	3.4	68.0%
AL2	9.5	3.0	31.6%
AS2	10.2	3.9	38.2%

Where, stiffness ratio is defined as $K_{initial}/K_{th}$

Both Specimens AL2 and AS2 exhibited a smaller amount of stiffness compared with their theoretical stiffness. In the elastic range, excessive shear deformation in the joint or loss of the anchorage of beam longitudinal bars was reflected in the rapid degradation of the stiffness. When the beam longitudinal bars yielded, the all specimens experienced a significant loss of stiffness while it was observed that the presence of a slab enhanced the initial stiffness significantly. The initial stiffness of Specimens AS1 and MAS1 were about 55% higher than that of Specimen AL1 and when comparing the initial stiffness of Specimen AS2 with Specimen AL2, the slab contributed to the increase in stiffness by up to 30%.

The peak-to-peak stiffness versus horizontal displacement for the specimens at each storey drift level is shown in **Figures 5-59** and **5-60**. It can be seen from the figures that the theoretically calculated stiffness was overestimated which might be due to the ignorance of the initial damage prior to the test. All specimens experienced stiffness degradation with the applied loading and when the deformation level was reached between the storey drift ratios of 1.0% to 1.5%, all specimens had a significant decrease in the global tangent stiffness due to the failure of a critical member. In the final stages, the stiffness continuously decreased to approximately 10% of the predicted elastic stiffness but the specimens with a slab exhibited higher stiffness than their counterparts although the stiffness values at the final stage were quite close.

Figure 5-55 Comparison of storey shear force versus horizontal displacement relationship among Specimens AL1, AS1, and MAS1

Figure 5-56 Comparison of storey shear force versus horizontal displacement relationship between Specimens AL2 and AS2

Figure 5-57 Comparison of storey shear force versus horizontal displacement relation between Specimens AL1 and AL2

Figure 5-58 Comparison of storey shear force versus horizontal displacement relationship between Specimens AS1 and AS2

Figure 5-59 Stiffness degradation in Specimens AL1, AS1 and MAS1

Figure 5-60 Stiffness degradation in Specimens AL2 and AS2

5.6.3 Joint Shear

The following expression was used to determine joint shear force,

$$V_t = A_s f_s + A'_s f'_s - V_{col} \quad (5-1)$$

where, V_t is the total joint shear force, A_s and A'_s are the areas of the tension and compression bars in the beam, f_s and f'_s are the stresses in the tension and compression bars in the beam and V_{col} is the column shear force.

Due to the absence of joint transverse reinforcement in all the specimens except Specimen MAS1, the total joint shear force was carried by the joint concrete. The concrete shear cracking stress and ultimate shear strength of the joints are tabulated in **Table 5-4**. According to ACI 318 [A2], the shear cracking stress carried by un-reinforced concrete can be expressed as follow:

$$v_c = 0.29 \sqrt{f'_c} \sqrt{1 + 0.29 \frac{P}{A_g}} \quad (5-2)$$

where P is the axial load acted on the column, A_g is the gross area of the column cross section, and f'_c is the concrete compressive strength. In the current tests, there were no column axial loads were applied, therefore the theoretical shear cracking stress was $0.29 \sqrt{f'_c}$ indicating that ACI equation gives a reasonable estimate of the lower shear cracking stresses in Specimens AL1 and AL2. The results given in **Table 5-4** for Specimens AS1 and AS2 show that the joint concrete was confined to a certain extent by the presence of transverse beam and slab and as a result, the joint could carry more shear forces even though the diagonal cracks had occurred in the joint core.

In Specimen MAS1, the joint transverse reinforcement carried part of the joint shear force; however, none of the joint hoops reached the yield strain during the test.

Table 5-4 Measured shear cracking stress and ultimate shear strength

Specimen	Shear cracking $\sqrt{f'_c}$ (MPa)	Ultimate shear $\sqrt{f'_c}$ (MPa)
AL1	0.33	1.29
AS1	0.40	1.51
MAS1	0.46	1.43
AL2	0.31	1.06
AS2	0.42	1.05

5.6.4 Member Contribution to Horizontal Displacement

The contributions of different members to specimen deformation give a good understanding of the behaviour of the specimens and help in visualizing the possible failure modes. In general, at a large deformation level, an increase in the participation of the most damaged members was accompanied by a serious loss of strength. For the specimens with a strong beam-weak column system, the column deformation contributed more to the lateral displacement during the test and the specimens could be considered to have failed due to the column flexural failure accompanying with the joint serious distress. The behaviour of Specimens AL2 and AS2 showed that the failure mode in these specimens was replaced by a weak beam-strong column system. For example, in the case of Specimen AS2, the beam member initially contributed 25% of the lateral deformation and it steadily increased to 60%, at the same time the column contribution remained constantly throughout the test, whereas the joint contribution decreased with the increasing of storey drift ratio.

5.6.5 Energy Dissipation

During a major earthquake, a tremendous amount of energy is released. If the building is designed to withstand the earthquake without collapse, it must be capable of absorbing and dissipating energy. The energy dissipating capability of a structure is very important because it has a strong influence on the response of the structures under dynamic loadings.

Figure 5-61 Comparison of energy dissipation among Specimens AL1, AS1 and MAS1

Figure 5-62 Comparison of energy dissipation between Specimens AL2 and AS2

For each specimen, the energy dissipated in each loading cycle was obtained by calculating the area enclosed by the corresponding load-displacement hysteretic loop. **Figures 5-61 and 5-62** illustrate the energy dissipated at different ductility levels for each specimen. Generally, in the elastic range, the specimens with a slab presented a similar energy dissipation capacity when compared to their counterparts. Once the loading cycle ran into the inelastic range, the energy dissipation gradually increased due to the presence of a slab. The energy dissipation capacities of Specimens AS1 and MAS1 rose up to 37.5% and 32.6%, respectively, whereas for Specimen AS2, the maximum enhancement of energy dissipation capacity was 49%.

5.7 Summary

In this chapter, the seismic performance of the lightly reinforced beam-column joints was evaluated through experimental and analytical approaches. Based on these studies, some conclusions can be summarized as follows:

- (1) In general, the test specimens could maintain their strength before a storey drift ratio of 2.0% and after that strength and stiffness degradation would occur. Amongst the five specimens, the performance of Specimen AL1 was very poor. This was mainly due to the insufficiency of joint transverse reinforcement and the inadequate anchorage of the beam longitudinal bars in the joint core.
- (2) The column-to-beam flexural strength ratio significantly affected the global as well as the local behaviour of the specimens. When the specimen was designed as a “strong beam-weak column” connection, the deformation demand on the column was significant. It might result in a large magnitude of contribution to the horizontal displacement and yielding in the column bars. Once the ratio of the column-to-beam flexural strength increased such as in Specimens AL2 and AS2, the contributions due to the column and beam deformation were quite close.

- (3) The maximum nominal horizontal joint shear stress of specimens in the first group exceeded the design code limit and therefore, the joint failure mechanism could be expected in these test specimens. After diagonal tension cracking, tension strains prevailed in the beam and column bars in the joint core resulting in significant joint expansion. Furthermore, the participation of slab bars induced additional shear forces into the joint; therefore, it was important to provide shear reinforcement in the joint to resist shear forces.
- (4) The presence of a slab influenced the behaviour of the specimens with increasing the load-carrying capacity of the main beam and imposing torsional moment in the transverse beam and then providing additional confinement to the joint.
- (5) Previous studies have indicated that, in interior beam-column joints, the effective slab width would vary from 1 to 5 times the beam depth on each side of the beam. It was found in this research that the development of effective slab width is correlated to the flexural strength ratio, and the global displacement demand. In Specimen AS1 and MAS1, the effective slab width did not develop because the overall performance was governed by the column behaviour. In Specimen AL2 and AS2, due to the “strong column-weak beam” configuration, the specimens were expected to be governed by the beam behaviour as such a effective slab width being of 1 times the beam depth on each side of the beam face plus beam width was found in Specimen AS2. However, such an effective slab width was obtained at a higher deformation level.
- (6) In the first group of the specimens, the ratio of the column depth to beam bar diameter did not satisfy the design limits by ACI 318 [A2], therefore tension strains were found in either the joint core or the compression zone of the beam and yielding penetration in the joint core would occur. Comparing the measured strains in the beam bars of Specimens AL1 and AL2, it was found that increasing the ratio of the column depth to beam bar diameter will improve strain distributions in the beam bars. However, in Specimen AS2, the beam top bars in the compression zone were also found in tension

during the test indicating that the ACI 318 [A2] limits may not be strict to prevent the slippage.

The Design limit by NZS 3101 [C1] is expressed as:

$$\frac{h_c}{d_b} \geq 0.3 \frac{\alpha_0 f_y}{\alpha_f \sqrt{f'_c}}$$

With assumptions that $\alpha_0 = 1.25$, $\alpha_f = 1.0$, $f_y = 460$ MPa and $f'_c = 40$ MPa. The NZS 3101 [C1] recommended column depth to beam bar diameter ratio is about 27 in this study. It is obvious that the NZS 3101 [C1] limit is much stricter than that of ACI 318 [A2]. However, the applicability of the NZS3101 [C1] limits on the non-ductile designed beam-column should be validated by extensive experimental investigations.

- (7) In Specimen MAS1, the provided transverse reinforcement in the joint core and the reduced spacing of the transverse reinforcement within the potential hinge regions could not improve the overall performance during the test. The measured strains in the joint hoops were very low and therefore, the joint shear resistance mechanism was mainly attributed to the diagonal concrete compression strut mechanism. The role of the joint transverse reinforcement in this specimen was to provide joint confinement rather than enhance of the truss mechanism due to the worsening bond condition in the joint.

Chapter 6

Test Results of Interior Wide Beam-Column Joints

6.1 Introduction

In order to examine the seismic behaviour of wide beam-column joints, a series of experimental investigations were carried out. Altogether, six wide beam-column connections were prepared and tested under reversed cyclic loadings. The test specimens contained three interior and three exterior wide beam-column connections. This chapter describes the test results of the interior wide beam-column joint specimens associated with the analytical results to provide some information on understanding the behaviour of wide beam-column joints.

6.2 Test Results of Specimen IWB-1

The ratio of the column flexural strength to beam flexural strength and the load-carrying capacity of Specimen IWB-1 were estimated by using a reinforced concrete sectional analysis software Response-2000. The minimum ratio of the column to beam flexural strength was larger than 4.0 and as a result plastic hinges were expected to form in the beam. The ratio of the column depth to beam bar diameter was $h_c/d_b = 40/1$, meanwhile the ratio of the beam depth to column bar diameter was $h_c/d_b = 12/1$. To predict load-carrying capacities of the specimens, it was assumed that the beam longitudinal bars at the section of the column face yielded simultaneously. Such a calculated capacity of Specimen IWB-1

was 249 kN in either loading direction. The following sections report the test results of the specimen.

6.2.1 General Observation

Cracking patterns were observed at different storey drift levels and shown in **Figure 6-1**. Flexural cracks, which were unsymmetrical in the wide beam, appeared early at a storey drift ratio of 0.25%. These cracks only concentrated in the left-beam until a storey drift ratio of 0.5% was reached. In the subsequent loading cycles with up to a 3% storey drift ratio, the beam flexural cracks extended vertically and spread along the beam.

At a storey drift ratio of 1.0%, a diagonal tension crack was observed in the side face of the beam, which started from the up-left corner of the transverse beam section and extended downwards to the bottom of the beam and subsequently, the diagonal cracks occurred increasingly and extended into the wide beams. Once the diagonal cracks went into the beams, they were developing horizontally along the beam longitudinal bars indicating that the concrete cover was spalling. The maximum crack width was 4 to 5 mm.

Figure 6-1 Observed cracking of Specimen IWB-1 at each selected storey drift level

When the specimen was loaded into a storey drift ratio of 2.5%, minor inclined cracks initiated in the column faces close to the wide beam. It is likely that these were the extension of the diagonal cracks in the joint panel. No flexural cracks were found in the column at this stage. When the specimen was finally loaded towards a storey drift ratio of 4%, the maximum crack width was 1cm. The beam concrete at the bottom face started to spall and resulted in the exposure of the beam longitudinal bars and transverse reinforcement. At the same time, the column concrete was crushed and spalled as well. **Figure 6-2** presents a photograph of Specimen IWB-1 after the test.

Figure 6-2 Photograph of Specimen IWB-1 after testing

6.2.2 Hysteretic Behaviour

Figure 6-3 illustrates the storey shear force versus horizontal displacement relationship for Specimen IWB-1. The theoretical capacities are also indicated in this figure. The specimen attained its theoretical capacity at a storey drift ratio of 1.0 % in either loading direction. Up to the final loading cycle, the specimen retained its load-carrying capacity without evident strength degradation. The maximum capacities, which were 277 kN in the positive and 268

kN in the negative loading direction, were obtained at a storey drift ratio of 1.5% and indicated that the specimen took over-strength factors at 11.2% and 7.6%, respectively.

The specimen experienced a decrease in the stiffness as the loading was applied. Due to the cracking of the concrete and yielding of the bars, the specimen carried lower loads during the repeated cycles at each loading level. A slight pinching behaviour was detected at early drift stages, but it was not significant until a large storey drift ratio of 2.0% was reached. Due to this pinching, the stiffness of the specimen dropped increasingly as a result. Until the end of the test, the residual load-carrying capacity of the specimen was 224 kN.

Figure 6-3 Storey shear force versus horizontal displacement relationship for Specimen IWB-1

6.2.3 Beam Behaviour

6.2.3.1 Strains in Beam Longitudinal Bars

Four top bars and four bottom bars were gauged respectively. The gauged bars on the top layer were labeled as A through D. Strain gauges on the bottom bars were located at the same locations corresponding to the gauge positions on the top bars and they were

numbered from A' through D'. The distribution of strain gauges on the beam bars is shown in **Figure 6-4**.

Figure 6-4 Distribution of strain gauges on the beam bars of Specimen IBW-1

Figures 6-5 and **6-6** illustrate the strain profiles of the beam longitudinal bars which passed through the column core. The diameters of these bars were 22 mm on the top layer and 20 mm on the bottom layer, respectively. Since the interior beam-column joints behaved with a symmetrical response in both the positive and negative loading directions, in most cases thereby, only the results corresponding to a single loading direction are discussed in this chapter.

From **Figure 6-5**, it can be seen that on the side in which the beam was subjected to a hogged moment, the top bars reached the yield strain at a storey drift ratio of 1.0% and in the subsequent loading cycles, yielding extended into the column core to quite some distance. On the opposite side, the top bars were under compression with compression stresses progressively increase as the loading ran. However, the compression yield was reached as far as a storey drift ratio of 2.0% being reached. In the tension region of the bottom bars, the tension yield was gained at a storey drift ratio of 1.0%, however, on the beam compression side; the compression yield did not arrive. It is found from **Figure 6-5** that in the beam compression zone, strains in both the top and bottom bars remained in compression indicating that the beam bars passing through the column core were well anchored.

(a) Beam top bar-A

(b) Beam bottom bar-A'

Figure 6-5 Strain profiles of beam longitudinal bars-A and A' of Specimen IWB-1

Figure 6-6 illustrates the strain profiles of the beam longitudinal bars B and B'. These bars showed a similar strain behaviour as that of the beam bars A and A'. In the top bar B, the tension yield was gained at a storey drift ratio of 1.0%. On the beam compression side, the compression yield was reached until the specimen was loaded to a storey drift ratio of 2.0%. In the bottom bar, the tension and compression yields were also occurred at 1.0% and 2.0% storey drift ratios, respectively. A good bond condition could also be found from the measured strains in these bars.

Figures 6-7 and **6-8** illustrate the strain profiles of the beam longitudinal bars C (C') and D (D'), which were placed in the outside beam portions. Strains in these bars distributed in a different manner compared with those passing through the column core.

Both the top and bottom bars placed in the outside beam portions did not yield during the test. Within the measured region, the bars in the beam compression zone developed compression strains only at early loading stages. As the loading increased, tension strains were monitored when the beam was not only in tension but also in compression, which indicated that the bond slip had occurred in these bars. This might attribute to the insufficient confinement of the concrete and the concrete spalling.

6.2.3.2 Strains in Beam Transverse Reinforcement

Four sets of beam transverse reinforcement, which was 50 mm and 350 mm away from the column faces, were gauged (see **Figure 6-11**). The recorded data showed that strains in the beam transverse reinforcement were quite low, in which the maximum value was only about 1500 $\mu\epsilon$. This indicated that the nominal shear stresses in the beam cross sections were very low because of the broad section area of the beam. **Figure 6-9** illustrates the strain profiles of the beam transverse reinforcement near the column face.

(a) Beam top bar-B

(b) Beam bottom bar-B'

Figure 6-6 Strain profiles of beam longitudinal bars-**B** and **B'** of Specimen IWB-1

(a) Beam top bar-C

(b) Beam bottom bar-C'

Figure 6-7 Strain profiles of beam longitudinal bars-C and C' of Specimen IWB-1

(a) Beam top bar-D

(b) Beam bottom bar-D'

Figure 6-8 Strain profiles of beam longitudinal bars-D and D' of Specimen IWB-1

Figure 6-9 Strain profiles of beam transverse reinforcement of Specimen IWB-1

6.2.3.3 Beam Rotation Behaviour

Figure 6-10 illustrates the beam rotation estimated from the transducers over a distance, which was equivalent to about $2h_b$, where h_b is the depth of the beam. The rotation was measured based on the readings of pairs of LVDT distributed along the beams. The rotation presented herein referred to the average rotation at the beam sections located 70 mm and 195 mm away from the column faces.

A gradual increase in the beam rotation was observed during the test. In the positive loading direction, the maximum rotation estimated over the region nearest the column face reached 0.008 rad. on the left beam and due to the unsymmetrical reinforcing details, the right beam reached the rotation at 0.011 rad. When the specimen was subjected to a negative loading, the maximum rotation of the left beam increased significantly and finally reached 0.013 rad. Meanwhile, the rotation of the right beam was 0.011 rad. For those sections away from the column faces, the rotation values were relatively lower at no more than 0.005 rad.

(a) Positive loading direction

(b) Negative loading direction

Figure 6-10 Measured beam rotation of Specimen IWB-1

Normally, the calculated rotation from the readings of the first pair of LVDT mounted on the beams nearest the column faces could be taken as the beam fixed-end rotation. Due to the relatively higher rotation values obtained, the beam fixed-end rotation and flexural deformation contributed significantly to the horizontal displacement. This can be found in **Figure 6-14** in the later section. At the loading stage corresponding to the yielding of the beam longitudinal bars, the measured beam fixed-end rotation θ_y was about 0.002 rad. where θ_y is defined as yield rotation. The measured yield rotation and maximum rotation are used to characterize the rotational ductility of elements in flexure, which is defined as:

$$\mu_\theta = \theta_{\max} / \theta_y \quad (6-1)$$

The rotational ductility factors of the beam of Specimen IWB-1 are shown in **Table 6-1**.

Table 6-1 Rotational ductility factors of Specimen IWB-1

Specimen IWB-1	μ_θ^+	μ_θ^-
Left wide beam	4.5	4.4
Right wide beam	3.7	4.8

6.2.4 Column Behaviour

6.2.4.1 Strains in Column Longitudinal Bars

The distribution of strain gauges on the column bars is shown in **Figure 6-11**. The gauged column longitudinal bars were labeled as E and F. Since the corner bars will suffer a large magnitude of tension or compression stresses, only the strains in these two bars were measured during the test. The strain profiles of the column longitudinal bars are shown in **Figure 6-12**. The respective measured yield strain was $2614\mu\epsilon$.

During the test, strains in the column longitudinal bars increased gradually with the applied loading. However, strains in the column bars for both the upper and lower columns did not reach the yield strain as such it can be concluded that the column of Specimen IWB-1

remained essentially in the elastic range up to the end of the test. This might due to the over-reinforcing of the column. The relatively lower strains also resulted in a small scale of the column fixed-end rotation and flexural deformation. The contribution due to the column deformation to the horizontal displacement can also be found in **Figure 6-14**.

As far as the final loading cycle was completed, there were no tension strains found when the column was in compression indicating that the column longitudinal bars were well anchored during the test.

Figure 6-11 Distribution of strain gauges on the column bars of Specimen IWB-1

6.2.4.2 Strains in Column Transverse Reinforcement

Four sets of column transverse reinforcement were gauged during the test but the recorded data showed that the column transverse reinforcement suffered a low level strain demand with the maximum value being about $182\mu\epsilon$. This indicated that the nominal shear stresses in the column cross sections were quite low during the test

(b) Column bar-E

(b) Column bar-F

Figure 6-12 Strain profiles of column longitudinal bars-E and F of Specimen IWB-1

6.2.4.3 Column Rotation Behaviour

Up to the final loading stage, a gradual increase in the column rotation was observed. At each loading cycle, the measured column rotation was approximately equal to 25% of the beam rotation. The maximum fixed-end rotation of the upper column was about 0.002 rad. in both loading directions. Meanwhile, the maximum fixed-end rotation of the lower column was 0.0015 rad. in the positive loading direction and 0.001 rad. in the negative loading direction, respectively. Comparing with the beam rotation response, the column rotation was on a very small scale. This might be due mainly to the relatively large stiffness of the column and it also indicated that the column behaviour was still in its elastic range.

6.2.5 Joint Behaviour

Due to the difficulty in measuring of joint shear deformation, the contribution of the joint shear deformation to the horizontal displacement could not be calculated.

As mentioned in Chapter 4, the joint core of Specimen IWB-1 was confined by using three sets of the column transverse reinforcement. The transverse reinforcement was fabricated using the T16 deformed bars which had the same properties as the T16 longitudinal bars in the wide beams and they were gauged to measure the joint transverse reinforcement strains during the test.

Figure 6-13 illustrates the strain profiles of the joint transverse reinforcement. Joint transverse reinforcement on the top and bottom layers suffered larger strain demands than the mid-layer one did in this specimen. However, all the joint transverse reinforcement did not reach the yield strain with the maximum value of 831 $\mu\epsilon$ being detected on the bottom layers.

Figure 6-13 Strain profiles of joint transverse reinforcement of Specimen IWB-1

6.2.6 Decomposition of Horizontal Displacement

Figure 6-14 illustrates the components of horizontal displacement at the peak of every loading run. These components were calculated following the procedure described by Hakuto [H1] and are shown here as percentages of the total horizontal displacement.

During a loading cycle up to a 2% storey drift, the main sources of the horizontal displacement were some unexpected and unmeasured components which might have been due to the joint shear deformation. During these stages, the unaccounted components contributed more than 50% to the total horizontal displacement, meanwhile, the column and beam flexural deformation plus column and beam fixed-end rotation contributed to the remaining displacement. Among them, contributions from the column fixed-end rotation and flexure deformation could be ignored.

After the beam longitudinal bars yielded, the components due to the beam fixed-end rotation and flexural deformation increased significantly. Up to a loading cycle of 4%

storey drift, the beam fixed-end rotation and flexural deformation contributed to the total horizontal displacement at percentages of 35.3% and 43.9%, respectively.

With reference to **Figure 6-14**, it can be seen that the contributions due to both the column fixed-end rotation and flexural deformation increased associated with the applied loading but these increases were limited. The maximum percentages of the column flexural deformation and fixed-end rotation were only 7.8% and 5.1%, respectively. Until the final stage, an evident increase of the column deformation was not observed.

Figure 6-14 Percentage of deformation to the displacement of Specimen IWB-1

6.3 Test Results of Specimen IWB-2

As mentioned in Chapter 4, Specimen IWB-2 was not strictly a wide beam-column connection. However, due to the non-typical beam section, it was tested in this research program for comparison. Since the column of Specimen IWB-2 would be able to be bent about its weak axis, the column flexural strength was lower than that of Specimen IWB-1. The ratio of the column to beam flexural strength reduced to 1.5 but it still resulted in a

“strong column-weak beam” configuration. The maximum ratio of the column depth to beam bar diameter was 18.75, which was lower than the ACI 318 [A2] limits. The ratio of the beam depth to column bar diameter still was 12. As a result, the load-carrying capacity of Specimen IWB-2 was expected to be governed by the beam strength. The theoretical capacity of Specimen IWB-2 was 207 kN based on the measured material properties.

Figure 6-15 Observed cracking of Specimen IWB-2 at each selected storey drift level

6.3.1 General Observation

Cracking patterns were observed at different storey drift levels and shown in **Figure 6-15**. In the wide beam and column, flexural cracking appeared early at a storey drift ratio of 0.5% and up to a storey drift ratio of 1.0%, beam flexural cracks extended vertically and spread along the beam span with full-depth cracks being formed at this stage. Meanwhile, full-width cracks were detected in the column face and a corner-to-corner diagonal tension crack was observed in the joint region. In the subsequent loading cycles up to a 3.0% storey drift, the beam flexural cracks spread along the full beam span length. From **Figure 6-15**, it can be seen that flexural cracks were evenly spaced in the wide beam but there were no shear cracks observed during the test.

In the column, the flexural cracks spread vertically and covered almost three-fourths of the column height. Within the joint region, two main diagonal cracks developed resulting in the generation of diagonal shear planes in two directions.

Figure 6-16 Photograph of Specimen IWB-2 after testing

As far as the specimen was loaded to a storey drift ratio of 3%, concrete covers spalled from the column. **Figure 6-16** presents a photograph of Specimen IWB-2 after the test.

6.3.2 Hysteretic Behaviour

Figure 6-17 illustrates the storey shear force versus horizontal displacement relationship for Specimen IWB-2. Theoretical capacities are also showed in this figure. The specimen attained its theoretical capacity approximately at a storey drift ratio of 1.0% in either loading direction and, at a storey drift ratio of 1.5%, the specimen gained its maximum capacity at 215 kN in the positive loading direction indicating a limited over-strength factor at 3.8%. After this point, the strength of the specimen decreased as the test was processed and at the final stage, the residual load-carrying capacity was 164 kN.

In the negative loading direction, the strength decreased constantly after the theoretical capacity was reached at a 1% storey drift ratio. When the test was completed, the residual load-carrying capacity in this loading direction was 114.7 kN indicating that the specimen had lost its load-carrying capacity of 45%.

Figure 6-17 Storey shear force versus horizontal displacement relationship for Specimen IWB-2

The hysteretic loops indicated that the specimen experienced evidently strength and stiffness degradation than that of Specimen IWB-1 as the loading was applied to the specimen. A pinching behaviour was also detected since the beam bars started to yield. The loss of load-carrying capacity enhanced the stiffness degradation as a result.

6.3.3 Beam Behaviour

6.3.3.1 Strains in Beam Longitudinal Bars

The distribution of strain gauges on the beam bars of this specimen was different from that of Specimen IWB-1. Three top bars were gauged with five strain gauges attached to each bar, whereas on the corresponding bottom bars, three strain gauges were used to measure the strain responses. Strain gauges on the top layer were labeled as A to C and the corresponding gauges on bottom layer were labeled as A' to C'. The distribution of the strain gauges is shown in **Figure 6-18**.

Figure 6-18 Distribution of strain gauges on the beam bars of Specimen IWB-2

Figure 6-19 to Figure 6-21 illustrate the strain profiles of the beam longitudinal bars on the top and bottom layers. All the measured beam top and bottom bars behaved with similar strain behaviour. As the test proceed, strains in the beam top and bottom longitudinal bars increased progressively but did not reach the yield strain up until a storey drift ratio of 2.0% was reached. Moreover, only the tension yield was found during the test. In most of cases, the beam bars in the compression zone remained in compression, which indicated a good bond condition in the joint core. The exceptions were the top bar B and bottom bar C'. Top bar B remained in compression at a storey drift ratio of 0.5% and, as the loading increased, the strains shifted into tension but the magnitude of the tension strains was not very higher. In the bottom bar C', tension strains were also found in the beam compression zone as the specimen was loaded beyond a storey drift ratio of 0.5%.

The strain profiles of the beam top bar B and bottom bar C' indicated that the bond slip had occurred in these two bars. It should be noted that the diameters of the bars B and C' were 20 mm and 16mm respectively, which led to column depth to bar diameter ratios at 15 and 18.75. In addition, the wide beam was also reinforced with T22 bars leading to the lowest column depth to the bar diameter ratio at 13.6. Since the anchorage limit required by ACI-ASCE 352 [A3] was not satisfied in the current design of the wide beam, the bond deterioration was expected to happen as a result.

Strains in the outmost bars C and C' were slightly higher than those in the interior bars, but the differences among these bars were not significant indicating that the beam sections remained in plane as the beam deformed.

6.3.3.2 Strains in Beam Transverse Reinforcement.

Similarly, four sets of beam transverse reinforcement, which were 50 mm and 350 mm away from the column faces, were gauged. The maximum strain was about 126 $\mu\epsilon$ indicating that nominal shear stresses in the beam cross sections were very low as a result of

the broad section area of the beam. **Figure 6-22** illustrates the strain profiles of the beam transverse reinforcement near the column face.

6.3.3.3 Beam Rotation Behaviour

Figure 6-23 illustrates the beam rotation under both the positive and negative loading directions. A gradual increase in the beam rotation was observed during the test. Since yielding of the beam bars was not observed from the measured strains until a large deformation was reached, the yield rotation “ θ_y ” of Specimen IWB-2 was defined as the rotation obtained at the moment, when the theoretical capacity was reached.

Table 6-2 Rotational ductility factors of Specimen IWB-2

Specimen IWB-2	μ_{θ}^{+}	μ_{θ}^{-}
Left wide beam	2.5	3.4
Right wide beam	3.3	3.1

Comparing the rotation response of Specimen IWB-2 with that of Specimen IWB-1, it was found that, at early deformation stages, the magnitude of beam rotation for these two specimens was relatively close; but at large deformation stages, the beam rotation of Specimen IWB-2 was lower than that of Specimen IWB-1 and as a result, the rotational ductility factors of Specimen IWB-2, which were calculated by **Equation 6-1**, were slightly smaller than those of Specimen IWB-1. **Table 6-2** tabulates the maximum rotation ductility factors of Specimen IWB-2, which were obtained at a storey drift ratio of 4.0%.

(a) Beam top bar-A

(b) Beam bottom bar-A'

Figure 6-19 Strain profiles of beam longitudinal bars-A and A' of Specimen IWB-2

(a) Beam top bar-B

(b) Beam bottom bar-B'

Figure 6-20 Strain profiles of beam longitudinal bars-**B** and **B'** of Specimen IWB-2

(a) Beam top bar-C

(b) Beam bottom bar-C'

Figure 6-21 Strain profiles of beam longitudinal bars-C and C' of Specimen IWB-2

Figure 6-22 Strain profiles of beam transverse reinforcement of Specimen IWB-2

6.3.4 Column Behaviour

6.3.4.1 Strains in Column Longitudinal Bars

The distribution of strain gauges on the column bars is shown in **Figure 6-24**. **Figure 6-25** illustrates the strain profiles of the column longitudinal bars which were labeled as D and E.

Unlike Specimen IWB-1, strains in the column longitudinal bars of this specimen increased evidently as the test proceeded. However, the column bars did not yield in either loading direction indicating that the column of Specimen IWB-2 remained essentially in the elastic range until the end of the test.

In the column compression zone, the bars suffered tension strains but the magnitude of the tension strains was very lower (see **Figure 6-25**). This indicated a worsened anchorage condition and the bond slip being occurred during the test. Since the column of the test specimen was reinforced with T25 bars, the beam depth to column bar diameter ratio,

which was only 12, was very much lower than the ACI 352 [A3] recommendation and as a result, the bond deterioration in the column bars would occur.

(a) Positive loading direction

(b) Negative loading direction

Figure 6-23 Measured beam rotation of Specimen IWB-2

Figure 6-24 Distribution of strain gauges on the column bars of Specimen IWB-2

6.3.4.2 Strains in Column Transverse Reinforcement

Four sets of column transverse reinforcement were gauged during the test. However, the recorded data showed that the column transverse reinforcement suffered a low level strain demand with the maximum value being about $250\mu\epsilon$ indicating that the nominal shear stresses in the column cross sections were very low during the test.

6.3.4.3 Column Rotation Behaviour

Up to the final loading stage, column rotation gradually increased as the lateral force applied to the specimen. When the specimen was subjected to the final deformation stage, the maximum fixed-end rotation was 0.006 rad. for the upper column and 0.005 rad. for the lower column, respectively. Comparing the column rotation response Specimen IWB-2 with that of Specimen IWB-1 at each loading level, it was found that the column rotation of Specimen IWB-2 was larger than that of Specimen IWB-1. This was because that the column of Specimen IWB-2 was more flexible than that of Specimen IWB-1. As a consequence, the deformation demand varied among the different members adjacent to the joint panel. The redistribution of the deformation demand could be reflected through the contributions of each member's deformation to the total horizontal displacement. This will be discussed in the following section.

(a) Column bar-E

(b) Column bar-D

Figure 6-25 Strain profiles of column longitudinal bars-**D** and **E** of Specimen IWB-2

6.3.5 Joint Behaviour

Joint Deformation

In the loading to $\pm 0.25\%$ storey drift ratio, there was a little joint shear distortion and expansion. When the loading was at $\pm 0.5\%$ storey drift ratio, in which the diagonal tension cracks were observed in the joint, the joint shear distortion increased rapidly and the respective joint shear distortion increased to approximately 0.12% . Subsequent loading cycles resulted in a consistent increase in the joint distortion as shown in **Figure 6-26** and the maximum joint shear distortion of 0.55% was obtained in the second cycle of the loading to a storey drift ratio of 4.0% . Comparing with the joint shear behaviour of the lightly reinforced conventional beam-column joints described in Chapter 5, it is believed that the joint shear distortion of the wide beam-column connections is much lower due to the large volume of joint core.

Figure 6-26 Measured joint shear distortion of Specimen IWB-2

Strains in Joint Transverse Reinforcement

Figure 6-27 illustrates the strain profiles of the joint transverse reinforcement. Strains in the top layer and bottom layer transverse reinforcement were quite low, whereas higher strains were found in the mid-layer transverse reinforcement with the maximum strain being measured at $750 \mu\epsilon$. All the joint hoops did not reach the yield strain when they subjected to both tension and compression stresses.

Figure 6-27 Strain profiles of joint transverse reinforcement of Specimen IWB-2

6.3.6 Decomposition of Horizontal Displacement

Figure 6-28 illustrates the components of horizontal displacement at the peak of every loading run. At early stages up to a storey drift ratio of 1.0%, the main sources of the horizontal displacement were the joint shear deformation. During these stages, the joint shear deformation component contributed more than 80% to the total horizontal displacement. However, as the specimen was loaded to a storey drift ratio of 2.0%, the contribution of the joint shear deformation decreased rapidly to the 35% of the total

horizontal displacement. At the final stage, the joint shear deformation contributing to the total horizontal displacement was only about 10%.

It has been shown that the contributions due to the beam fixed-end rotation and flexural deformation were limited during the early stages and, as a storey drift ratio of 2.0% was reached, the beam fixed-end rotation and flexural deformation made their maximum contributions to the total horizontal displacement. The magnitude of percentage of the displacement components due to the beam fixed-end rotation and beam flexural was 15% and 25%, respectively. It was worth noting that the beam longitudinal bars did not yield at this stage, therefore the large magnitude of beam fixed-end rotation was mainly attributed to the bond slip of the beam bars. At a storey drift ratio of 3%, the contributions due to the beam deformation were reduced, but they increased again with the occurrence of the yielding of the beam bars at a storey drift ratio of 4%.

Figure 6-28 Percentage of deformation to the displacement of Specimen IWB-2

The contributions due to the column deformation were much lower at early stages. The column fixed-end rotation and flexural deformation evidently increased when a storey drift

ratio of 2% was reached. In the remaining loading cycles, the contributions of the column deformation increased progressively associated with the large magnitude of column deformation. From a storey drift ratio of 3% to 4%, the contribution of the column flexural deformation combined with the column fixed-end rotation accounted for 60% of the total horizontal displacement.

6.4 Test Results of Specimen IWB-3

As mentioned in Chapter 4, the difference between Specimens IWB-1 and IWB-3 was only the compression strength of the concrete. Based on the measured concrete strength of Specimen IWB-3, the theoretical capacity of Specimen IWB-3 was 244 kN in either loading direction. This value was slightly lower than that of Specimen IWB-1, which was 249 kN. It can be expected that the influence of concrete strength is not significant on the overall performance of the test specimens

6.4.1 General Observation

Cracking patterns were observed at different storey drift levels as shown in **Figure 6-29**. Generally, crack patterns of Specimen IWB-3 were similar to those of Specimen IWB-1. In the wide beam of Specimen IWB-3, flexural cracking initiated at a storey drift ratio of 0.25% in both loading directions. At a storey drift ratio of 1.0%, beam flexural cracks extended vertically and spread along the beam span with full-depth cracks generating in both of the beams. At the same time, diagonal cracks were opening, which started from the corner of the transverse beam section and extended towards the beam face. When the specimen was loaded to a storey drift ratio of 1.5%, inclined cracks were observed in the upper column where close to the wide beam. In the subsequent loading cycles up to a 3% storey drift ratio, the beam flexural cracks spread over three-fourths of the beam span, however, only the flexural cracks were found in the beams and the spacing of these cracks was evenly. Shear cracking did not generate until the end of the test.

Figure 6-29 Observed cracking of Specimen IWB-3 at each selected storey drift level

Figure 6-30 Photograph of Specimen IWB-3 after testing

As far as a storey drift ratio of 2.0%, the diagonal cracks were generated increasingly and extended into the wide beam with a main corner-to-corner diagonal crack being formed at this stage. Similar to Specimen IWB-1, when the diagonal cracks went into the beams, they extended horizontally along the beam longitudinal bars. At this moment, minor inclined

cracks appeared on both the upper and lower column faces in addition to the flexural cracks being found in the lower column. When the specimen was loaded to a storey drift ratio of 3.5%, the beam concrete cover started to spall and the beam longitudinal bars and transverse reinforcement were exposed and could be seen while the maximum crack width changed to 1.5 cm. **Figure 6-30** presents a photograph of Specimen IWB-3 after the test.

6.4.2 Hysteretic Behaviour

The storey shear force versus horizontal displacement relationship is shown in **Figure 6-31**. The specimen attained its theoretical capacity at a storey drift ratio of 1.0% in either loading direction. The maximum capacity was obtained at a storey drift ratio of 1.5% with the value being about 256 kN in the positive loading direction indicating an over-strength factor at 4.9%. Also at a storey drift ratio of 1.5%, the maximum capacity of 248 kN was gained in the negative loading direction indicating the over-strength effect was limited in this loading direction.

Figure 6-31 Storey shear force versus horizontal displacement relationship for Specimen IWB-3

In the remaining loading cycles, the specimen lost its load-carrying capacity; however, the strength degradation was not significant. From the hysteretic loops, the stiffness degradation and pinching behaviour could also be found. When the test completely finished, the residual load-carrying capacity of the specimen was 220 kN

6.4.3 Beam Behaviour

6.4.3.1 Strains in Beam Longitudinal Bars

Specimen IWB-3 had the same reinforcing details like Specimen IWB-1, therefore, for Specimen IWB-3, the distribution of strain gauges on the beam longitudinal bars was identical to that of Specimen IWB-1 and these strain gauges were designated with the same number (see **Figure 6-4**). Since strains in the beam longitudinal bars were shown in an approximately symmetrical manner, therefore only strain profiles in a single loading direction were presented in this section.

Figures 6-32 and 6-33 illustrate the strain profiles of the beam top and bottom bars which passed through the column core. It can be seen that strains in both the top and bottom bars reached the tension yield between a storey drift ratio of 1.0% to 1.5% and in the subsequent loading cycles, yielding extended into the column core to some distance. In the beam compression zone, compression strains progressively increased with the loading run, but the compression yield was reached only in the top bar B at a storey drift ratio of 2.0% with strains in both the top and bottom bars remaining in compression indicating that the bars passing through the column core were under a good bond condition.

Figures 6-34 and 6-35 illustrate the strain profiles of the beam top and bottom bars which were placed in the outside beam portion where the strains in these bars were distributed in a different manner compared to those passing through the column core.

Both the top and bottom bars placed in the outside beam portion did not yield during the test. Within the measured region, the bars in the beam compression zone were under

compression strains only at early loading stages. As the loading increased, tension strains were monitored when the beam was not only in tension but also in compression, which indicated that the bond slip had occurred in these bars. This might be due to the insufficient confinement of the concrete and beam bar anchorage.

6.4.3.2 Strains in Beam Transverse Reinforcement

Also four sets of beam transverse reinforcement which had the same locations like Specimen IWB-1 were gauged. The recorded data showed that strains in the beam transverse reinforcement were quite low with the maximum value being about $1300 \mu\epsilon$ indicating that the nominal shear stresses in the beam cross sections were very low as a result of the broad section area of the beam. **Figure 6-36** illustrates the strain profiles of the beam transverse reinforcement near the column face.

6.4.3.3 Beam Rotation Behaviour

A gradual increase in the beam rotation was observed during the test. In the positive loading direction, the maximum rotation estimated over the region nearest the column faces reached 0.011 rad. on the left beam and 0.014 rad. on the right beam, respectively, whereas when the specimen was subjected to the negative loading, the maximum rotation of the left beam was 0.013 rad. and the rotation of the right beam was 0.017 rad.. For those sections away from the column faces, the rotation values were relatively low at no more than 0.005 rad.. **Table 6-3** tabulates the calculated rotational ductility factors based on the measured θ_y and θ_u . Here, θ_y refers to the rotation at the stage when the theoretical capacity was reached; θ_u is the rotation at the terminated stage of the test. The rotational ductility factors of the wide beam in Specimen IWB-3 were slightly higher than those of Specimen IWB-1.

Table 6-3 Rotational ductility factors of Specimen IWB-3

Specimen IWB-3	μ_{θ}^{+}	μ_{θ}^{-}
Left wide beam	3.9	4.6
Right wide beam	5.0	6.0

(a) Beam top bar-A

(b) Beam bottom bar-A'

Figure 6-32 Strain profiles of beam longitudinal bars-A and A' of Specimen IWB-3

(a) Beam top bar-B

(b) Beam bottom bar-B'

Figure 6-33 Strain profiles of beam longitudinal bars-**B** and **B'** of Specimen IWB-3

(a) Beam top bar-C

(b) Beam bottom bar-C'

Figure 6-34 Strain profiles of beam longitudinal bars-C and C' of Specimen IWB-3

(a) Beam top bar-D

(a) Beam bottom bar-D'

Figure 6-35 Strain profiles of beam longitudinal bars-D and D' of Specimen IWB-3

Figure 6-36 Strain profiles of beam transverse reinforcement of Specimen IWB-3

6.4.4 Column Behaviour

6.4.4.1 Strains in Column Longitudinal Bars

The distribution of strain gauges on the column bars of Specimen IWB-3 is same as Specimen IWB-1.(see **Figure 6-11**) The strain profiles of the column longitudinal bars are shown in **Figure 6-37**.

Strains in the column bars increased gradually as the test proceeded, but yielding was not observed at any locations in the bars. The maximum strain was reached at $1274 \mu\epsilon$, which was 50% of the yielding value and therefore, the column of Specimen IWB-3 remained essentially in the elastic range until the end of the test.

Tension strains were observed in the compression zone indicating that the column longitudinal bars lost their anchorage during the test although the magnitude of tension strains was not significant.

(a) Column bar-E

Column bar-F

Figure 6-37 Strain profiles of column longitudinal bars-E and F of Specimen IWB-3

6.4.4.2 Strains in Column Transverse Reinforcement

Four sets of column transverse reinforcement were gauged during the test. However, the recorded data showed that the column transverse reinforcement suffered a low level strain demand with the maximum value being about $554\mu\epsilon$ indicating that the nominal shear stresses in the column cross sections were quite low during the test.

6.4.4.3 Column Rotation Behaviour

Similar to Specimen IWB-1, column rotation gradually increased as the lateral force applied to the specimen. Up to the final positive loading direction, the maximum rotation was about 0.0012 rad. In the negative loading direction, the rotation of the upper column was 0.0019 rad. Compared with the upper column, the lower column showed a slightly larger rotation demand, but the difference was not significant.

6.4.5 Joint Behaviour

Figure 6-38 illustrates the strain profiles of the joint transverse reinforcement. Similar to Specimen IWB-1, the joint transverse reinforcement of Specimen IWB-3 at the mid-layer attained the maximum strain at $750\mu\epsilon$. At the top and bottom layers, joint transverse reinforcement suffered lower strain demands compared to the mid-layer transverse reinforcement. However, all of the joint transverse reinforcement did not reach the yield strain.

6.4.6 Decomposition of Horizontal Displacement

Figure 6-39 illustrates the components of horizontal displacement at the peak of every loading run. Up to the end of the test, the main sources of the horizontal displacement were also the unexpected and unmeasured components including the joint shear deformation. Until the final stage, the unaccounted components had contributed to almost 40% of the total horizontal displacement. The column and beam flexural deformation plus column and beam fixed-end rotation contributed to the remaining displacement. Comparing the total

horizontal displacement components in Specimen IWB-3, it can be seen that, the contributions due to the column fixed-end rotation and flexural deformation were still limited, but they were higher than those in Specimen IWB-1.

The components due to the beam fixed-end rotation and flexural deformation increased progressively with the loading run. Up to a storey drift ratio of 4%, the beam fixed-end rotation and flexural deformation contributions to the total horizontal displacement were 25% and 15%, respectively, which were lower than those in Specimen IWB-1.

With reference to **Figure 6-39**, the contribution due to the column flexural deformation was also increased with the applied loading. The maximum percentage of the column flexural deformation was obtained at the final stage, which was about 15%. The contribution due to the column fixed-end rotation decreased as the storey drift ratio increased with the highest value being reached at initial stages, which was about 8% of the total deformation. At the final stage, the percentage reduced to 5% of the total deformation.

Figure 6-38 Strain profiles of joint transverse reinforcement of Specimen IWB-3

Figure 6-39 Percentage of deformation to the displacement of Specimen IWB-3

6.5 Three Dimensional Finite Element Evaluations on Interior Wide Beam-Column Joint Response

6.5.1 Introduction

In order to provide additional insight into the interior beam-column joint behaviour, a finite element evaluation was conducted by using DIANA [D3]. The finite element models were constructed to match the as-built dimensions and material properties. The theoretical background and modelling criterion, which were used in the analyses are presented in the previous chapters. **Figure 6-40** illustrates a typical analytical model and boundary conditions.

Figure 6-40 Analytical model and boundary conditions

6.5.2 Verification of Experimental and Analytical Results

6.5.2.1 Comparison of Global Behaviour

Global behaviour is compared in terms of the comparison of hysteretic behaviour. **Figures 6-41 to 6-43** illustrate the hysteretic responses obtained through finite element analyses and experimental investigations. These figures indicate that the overall behaviour between the analytical and experimental results is basically in good agreement.

The analytical models show that the maximum capacities were reached around a storey drift ratio of 1.0% to 1.5% and after that, the strength and stiffness degradation were obvious. In the all analytical hysteretic loops, significant pinching was observed when the storey drift level was high. These phenomena were similar to those observed during the test.

Figure 6-41 illustrates the comparison of hysteretic behaviour between Specimen IWB-1 and Model IWB-1. In the analysis, the theoretical capacity was reached as early as a 0.5% storey drift ratio, which was slightly earlier than the experimental investigation. In either

loading direction, the maximum capacity was gained as the model was loaded into a drift ratio of 1.0%, which was 308 kN in the positive loading direction and 286 kN in the negative loading direction, respectively. Therefore, the maximum capacities of Model IWB-1 were higher than the measured values in Specimen IWB-1 at 17.5% and 7.5%, respectively.

Figure 6-41 Comparison of hysteretic behaviour between Specimen IWB-1 and Model IWB-1

Figure 6-42 indicates that the hysteretic behaviour of Model IWB-2 was underestimated. Until a storey drift ratio of 2.5%, the experimental and analytical results were comparable, but at a storey drift ratio of 3.0%, the load-carrying capacities of Model IWB-2 were 149 kN in the positive and 124 kN in the negative loading directions, respectively. According to the experimental results, Specimen IWB-2 could retain its load-carrying capacities at 177 kN in the positive and 147 kN in the negative loading directions, respectively, which was 18% higher than Model IWB-2 at this stage.

Figure 6-42 Comparison of hysteretic behaviour between Specimen IWB-2 and Model IWB-2

Figure 6-43 Comparison of hysteretic behaviour between Specimen IWB-3 and Model IWB-3

Figure 6- 43 illustrates the comparison of hysteretic behaviour between Specimen IWB-3 and Model IWB-3. It can be seen that the analytical and experimental results were in good agreement.

6.5.2.2 Comparison of Strains in Beam Longitudinal Bars

The strain behaviour of the analytical models is shown in **Figures 6-44 to 6-50**. The bar labels are identical to those defined in the tests and can be referred to in **Figures 6-4, 6-18**. Since the measured strains in the beam longitudinal bars in Specimens IWB-1 and IWB-3 were quite similar, therefore only the analytical results of Model IWB-1 are presented in this section.

In general, the analytical models produced a similar strain distribution to the measured ones. In Model IWB-1, the calculated strains of the beam longitudinal bars indicated that the bars passing through the column core were subjected to greater strain demands than those passing through the outside beam portions, which had been proved in the corresponding experimental investigation. From **Figures 6-44 and 6-45**, it can be seen that the maximum strains occurred at the positions adjacent to the column faces and the calculated strain magnitude in the internal bars was much higher than the measured value. It was noted that strain gauges were attached to the beam bars with a wider spacing during the test and therefore, the measured strains could not precisely reflect real strain distributions in the beam bars.

The exterior bars went into tension in the beam compression zone when the model was loaded from a 0.5% to 1.0% storey drift ratio (see **Figure 6-46 and 6-47**). In the outmost bars, tension strains were evenly distributed within the anchorage region, which indicated a poor anchorage condition between the bars and surrounding concrete. At the moment when the bars passing through the outside beam portions considerably into tension, the extensive diagonal cracks were observed on the beam faces indicating the torsional failure of the transverse beams.

(a) Beam top bar-A

(b) Beam bottom bar-A'

Figure 6-44 Strain profiles of beam longitudinal bars-A and A' of Model IWB-1

(a) Beam top bar-B

(b) Beam bottom bar-B'

Figure 6-45 Strain profiles of beam longitudinal bars-B and B' of Model IWB-1

(a) Beam top bar-C

(b) Beam bottom bar-C'

Figure 6-46 Strain profiles of beam longitudinal bars-C and C' of Model IWB-1

(a) Beam top bar-D

(b) Beam bottom bar-D'

Figure 6-47 Strain profiles of beam longitudinal bars-D and D' of Model IWB-1

(a) Beam top bar-A

(b) Beam bottom bar-A'

Figure 6-48 Strain profiles of beam longitudinal bars-A and A' of Model IWB-2

(a) Beam top bar-B

(b) Beam bottom bar-B'

Figure 6-49 Strain profiles of beam longitudinal bars-B and B' of Model IWB-2

(a) Beam top bar-C

(b) Beam bottom bar-C'

Figure 6-50 Strain profiles of beam longitudinal bars-C and C' of Model IWB-2

In Model IWB-2, strains in the beam top as well as bottom longitudinal bars were quite close. The calculated yield strain was obtained at a storey drift ratio of 2.0% and the strain values in the bottom bars were higher than those in the top bars, which were also found in the test. In the beam compression zone, tension strains were not found during the analysis indicating a satisfied anchorage condition.

6.5.2.3 Comparison of Strains in Column Longitudinal Bars

The calculated strains in the column longitudinal bars are shown in **Figures 6-51 and 6-52**. Tension strains in the column bars increased evidently as the models were loaded from a storey drift ratio of 0.5% to 1.0%, but in Model IWB-1, strains did not increase considerably in the remaining loading cycles. In Model IWB-2, strains in the column bars increased progressively with the applied loading. In any cases, the yield strain was not reached in the column bars and this phenomenon had been validated by the test results. The analytical results also indicated that the strain behaviour was controlled by the column to beam flexural ratio.

6.5.2.4 Evaluation of Effective Beam Width

Strain distributions in the beam longitudinal bars can be used to evaluate the effective beam width in which the beam bars transfer the inter-forces to the column and participate to the moment resisting. **Figures 6-53 and 6-54** illustrate the tension strains in the beam longitudinal bars at the sections adjacent to the column faces of Models IWB-1 and IWB-3.

As mentioned earlier, in Models IWB-1 and IWB-3, the beam bars passing through the column core reached the yield strain as early as a storey drift ratio of 1.0%, whereas the bars outside of the column did not reach the yield strain until the end of the analysis. Within the portions which extended to 100 mm from the both column faces, the beam longitudinal bars reached the yield strain between storey drift ratios of 1.0% to 1.5%. Therefore, the effective beam width of Models IWB-1 and IWB-3 can be taken as the column width plus 100 mm at

each side of the column face. The bars within these regions can effective to resist bending moments and the bars outside these regions cannot transfer the inter-forces due to the torsional failure of transverse beam.

Figure 6-51 Strain profiles of column longitudinal bars-**E** and **F** of Model IWB-1

(a) Column bar-D

(b) Column bar-E

Figure 6-52 Strain profiles of column longitudinal bars-**D** and **E** of Model IWB-2

Figure 6-53 Strain variations in the beam top bars of Model IWB-1

Figure 6-54 Strain variations in the beam top bars of Model IWB-3

6.6 Discussion of Experimental and Analytical Results of Interior Wide Beam-Column Joints

6.6.1 Load-Carrying Capacity and Hysteretic Response

The storey shear force versus horizontal displacement relationships show that all of the interior wide beam-column specimens had adequate load-carrying capacities and amongst these, Specimens IWB-1 and IWB-3 presented limited over-strength behaviour with the maximum capacities being about 11.2% and 4.9% larger than the theoretical capacities, respectively. However, the obvious strength enhancement was not found in Specimen IWB-2. As far as the ductility behaviour was concerned, the yield displacement was reached between storey drift ratios of 1.0% to 1.5%, at which moment the beam bars reached the yield strain and as a result, all the test specimens could obtain a ductility factor of 2.0 without significant strength degradation.

It is worth to recall that the theoretical capacity was calculated based on the measured material properties and with the assumption that the tension yield can be expected in all bars simultaneously. However, according to the experimental and analytical results, only the bars within the effective beam width yielded and exceeded the yield strain tremendously. Therefore, the over-strength behaviour in Specimens IWB-1 and IWB-3 was mainly attributed to strain hardening in these bars and other sources, such as the confining effects of the transverse beam. As the deformation level past the yielding level, some of the beam longitudinal bars, particularly those passing through the column, achieved strain values well in excess of the yielding and at larger deformation levels, the strain hardening in these bars became more significant. This can be found in **Figures 6-5 and 6-6** or **Figures 6-32 and 6-33**. Due to the bars passing outside the column did not develop the tension or compression yield and therefore, the over-strength behaviour was not significant in these two specimens.

In Specimen IWB-2, the storey drift ratio at which the theoretical capacity was attained was relatively high, which was around a storey drift ratio of 1.5% or higher. This was mainly due to the flexibility of the specimen, which could be attributed to the lower stiffness of the shallow wide beam as well as the column. On the other hand, the early bond slip in both the column and beam longitudinal bars also made the specimen more flexible. Although the yield strain was finally reached, the strain values did not exceed the yield strain by much. During the test, stiffness degradation and hysteretic loops pinching, which is typical for reinforced concrete, were experienced by the specimen.

Comparing the test results of Specimen IWB-2 with those of Specimen IWB-1 or IWB-3, it is noticed that the lower beam to column width ratio of Specimen IWB-2 did not result in a noticeable improvement of the response. In contrast, Specimens IWB-1 and IWB-3 presented a better performance with respect to Specimen IWB-2. Therefore, the design recommendation on the beam to column width ratio cannot guarantee the performance by itself. Previous research has pointed out that the beam to column width ratio is not the most critical design parameter for wide beam systems.

As aforementioned, all the specimens met ACI-ASCE 352 [A3] recommendation on flexural ratio, which is 1.4 and therefore, a “strong column and weak beam” philosophy was always satisfied. Since the flexural ratio of Specimens IWB-1 and IWB-3 were much higher, columns in these two specimens were much stronger than the beams, and as a result it led to a low strain level in the column bars as well as the elastic response of the column during the test. From the strain profiles of the column longitudinal bars, it can be seen that the column bars in the all specimens did not reach the yield strain until the end of the test.

6.6.2 Comparison of Stiffness Degradation

Figure 6–55 illustrates the stiffness degradation for the interior wide beam-column joint specimens based on the measured results. In general, the stiffness of all these specimens

decreased dramatically when they were loaded from a storey drift ratio of 0.5% to 1.5% and beyond a storey drift ratio of 1.5%, the stiffness decreased slowly with the applied loading.

Amongst these specimens, Specimen IWB-1 presented the highest initial stiffness which was approximately 20 kN/mm at a storey drift ratio of 0.5%. Due to the cracks opening and bars yielding, the stiffness at a storey drift ratio of 1.5% only retained 35% of the initial one. From a storey drift ratio of 1.5% to the final stage, the decrease of the stiffness in each couple of the subsequent cycles was not significant.

The column of Specimen IWB-2 was more flexible than that of Specimen IWB-1 and as a result, Specimen IWB-2 presented the lowest initial stiffness at 14.5 kN/mm at a storey drift ratio of 0.5% which was equal to 72% of the initial stiffness of Specimen IWB-1 at this loading level. At a storey drift ratio of 1.5%, significant stiffness degradation was also observed, which was 36% of the original one.

The initial stiffness of Specimen IWB-3 was between Specimens IWB-1 and IWB-2. In the early stages and until a storey drift ratio of 1.0%, the magnitude of the stiffness of Specimen IWB-3 was quite close to that of Specimen IWB-2. However, as the loading level increased, the stiffness degradation trend and stiffness magnitude were identical to those of Specimen IWB-1.

It can be summarized that the influences of the column to beam flexural ratio and concrete strength with regards to the stiffness behaviour are reflected in the early stages. Once the concrete cracks opened extensively and the reinforcing bars yielded, these influences are not evident. At lower deformation levels, the deformation of the test specimens is sensitive to the concrete compressive strength. As a result, the stiffness of Specimen IWB-1 is slightly higher than that of Specimen IWB-3 and after the concrete cracks and bars yield, the difference of stiffness can be negligible.

Figure 6-55 Stiffness degradation in the interior wide beam-column joint specimens

6.6.3 Anchorage Conditions for the Beam and Column Longitudinal Bars

6.6.3.1 Beam Bars Passing through the Column and Outside the Column.

In Specimens IWB-1 and IWB-3, three of the nine beam top bars and bottom bars passing through the column led to anchorage ratios of 38% and 44%, at the same time, the column depth to beam bar diameter ratios were 41 for the T22 bars and 45 for the T20 bars, respectively. These values exceed the design recommendations by ACI 381[A2], therefore, a sufficient anchorage for these bars can be expected and has been validated by the experimental and analytical results.

The remainders of the beam longitudinal bars were placed in the outside beam portions. If the anchorage length was taken as the column depth, the column depth to beam bar diameter ratios were 45 for the top bars and 56 for the bottom bars, respectively. From this point of view, the anchorage of the beam bars passing through the outside beam portions was satisfied with the design provisions, but the experimental and analytical results showed that the beam bars within these portions undertook an insufficient anchorage. The observed

strain behaviour of the beam longitudinal bars in Specimens IWB-1 and IWB-3 indicated that the full-width plastic hinges in the wide beam could not be developed. The outmost bars were under a poor anchorage condition after the torsional cracks developing and concrete spalling.

Stresses in the beam longitudinal bars which were placed in the outside beam portions were expected to be transferred by means of the torsion moment within the transverse beam. The torsional deformation of the outside beam portions caused the additional rotation of the wide beam and resulted in higher torsional stresses, which caused severe cracking in the outside beam portions and a bond failure along the column side and beam intersections.

Figure 6-56 presents a photograph of cracking patterns on the side face of the outside beam portion.

Figure 6-56 Cracks in the outside beam portion

Due to the smaller torsional stiffness of the outside beam portions, the strength of the wide beam will decrease. In addition, the extensive cracking in the outside beam portions may result in the pinching in the hysteretic response and deterioration in the hysteretic energy absorption capacity. However, in the current test specimens, no significant deterioration of

the hysteretic response was observed until a large deformation level was reached. This was attributed to the deep column section and a large beam side area through which the torsional moment was transferred. Therefore, the torsional stiffness of the transverse member is a controlling parameter for the wide beam-column subassemblages. The transverse reinforcement in the outside beam portions are expected to ensure the beam longitudinal bars developing the sufficient anchorage and to increase the torsional stiffness of the outside beam portions after cracking. However, in these tests, there was no extra transverse reinforcement provided in the outside beam portions. Instead, beam transverse reinforcement in the transverse beam was used to provide the confinement to the concrete and to ensure the sufficient anchorage of the beam longitudinal bars.

The beam longitudinal bars of Specimen IWB-2 were fully passing through the column and so as a result the column depth to beam bar diameter ratios were 14 for the T22 bars, 15 for the T20 bars and 19 for the T16 bars, which were slightly lower than the design recommendations. The observed yielding patterns of the beam longitudinal bars of Specimen IWB-2 indicated that the development of full-width plastic hinges in the wide beam could be expected when the deformation level was high.

6.6.3.2 Column Bar Anchorage

All the columns were reinforced with T25 deformed bars, which resulted in the beam depth to column bar diameter ratio of 12 and that it was much lower than the design recommendations. During the test, strains in the column longitudinal bars were quite low as results of the large flexural strength ratio. However, in Specimens IWB-2 and IWB-3 tension strains in the column bars were found when the columns were in compression indicating the loss of anchorage in these bars. It can be concluded that in these two specimens the bond slip in the column bars could not be prevented even though the design recommendation was satisfied.

6.6.4 Joint Shear

Since all the specimens were designed as limited ductile connections, therefore, the joint shear expected to control the performance of the subassemblages. As mentioned in Chapter 4, all of the joint cores were reinforced with a limited amount of transverse reinforcement. According to the joint shear mechanisms, the imposed joint shear forces were expected to be resisted via the diagonal concrete compression strut mechanism and after the joint shear strength was reached, the severe joint damage could not be avoided.

During the test, neither strains nor cracking patterns indicated that the joint was undergoing shear problems. Although the shear cracks were observed in the joint, they generally opened very little and most of them were carried by the diagonal concrete compression strut mechanism during the early stages (see **Figure 6-57**). After the extensive joint shear cracks occurred, the transverse reinforcement was required to carry most of the joint shear forces. However, in the tests as well as in finite element analyses, strains in the joint transverse reinforcement maintained a lower level. This might be attributed to that the column longitudinal bars were helpful to carry partial joint forces and in addition, some areas of the wide beam, which wraps around the column, were also participated in resisting the joint shear forces.

Figure 6-57 Diagonal concrete compression struts in the joint core

For all the specimens, the maximum nominal horizontal joint shear stress was only $0.8\sqrt{f'_c}$. Although the current test results showed that the magnitude of joint shear stresses and joint hoop strains were quite low, however, it is unlikely that the column section can be significantly reduced because in the design of wide beam structures other issues are of

concerned such as flexural ratio, anchorage length and frame stiffness, also the role of joint transverse reinforcement is still significant. It should be noted that in the current test specimens, the beam strength and strain hardening in the beam longitudinal bars did not fully develop during the test due to the insufficient torsion strength of transverse beam. If the torsion strength is enhanced, the joint will undertake a large magnitude of shear forces and thus the design of joint panels for shear resisting in the wide beam structures should follow the same rules as for the conventional beam-column frame.

6.6.5 Previous Studies on Wide Beam-Column Joints with a Deep Column

In the experiments carried out by Hatamoto *et al* [H4], four wide beam-column specimens including a specimen, WF-4, which was designed with a deep column instead of a conventional rectangular column, were tested to evaluate the effect of the beam to column width ratio on the performance of the connections.

In their tests, all the specimens reached a large deformation angle at 5% without strength deterioration. The maximum capacities were 100 to 110% of the values calculated based on the beam yielding mechanism. The hysteretic loops for some specimens showed a noticeable bond slip after the deformation angle of approximately 1%. The initial beam bar yielding was observed between drift angles of 1% to 1.5% and after that the effective beam width started to increase gradually with the deformation angle increase. For the specimens with lower beam width ratios, the effective beam width ratios reached the actual beam width ratios between drift angles of 2%~3%. However, Specimen WF-4 did not reach the actual beam width ratio, even in the final loading cycle.

For Specimen WF-4, all the beam longitudinal bars did not reach the yield strain even at a drift angle of 4%. In this specimen, the largest stiffness was presented at initial beam bars yielding and no pinching in the hysteretic loops was observed even in the final loop. The larger area of the outside beam portion reduced the maximum torsional stress of Specimen WF-4 to 60% of those specimens with a conventional column section. Although Specimen

WF-4 also experienced slip cracking along the column side, however, no significant deterioration in the hysteretic loops was observed.

It was pointed out by Hatamoto [H4] in this study that the torsional stiffness of the transverse beam was a control parameter for the wide beam-column specimens. When adopting the torsion strength for design, sufficient transverse reinforcement in the outside beam portions should be provided. However, based on the limited test results, it is difficult to determine how much reinforcement is sufficient.

6.6.6 Effective Slab Width and Effective Wide Beam Width

The slab contribution to the negative bending moment capacity of wide beam frames is of concerned to researchers around the world. Since limited wide beam-column connection tests including floor slabs were carried out previously, it is still not known how much additional flexural capacity is provided by floor slabs. It is generally accepted that, slabs in a wide beam building, the slab participation may not be as significant as in a non-wide beam building [G2]

However, a contrary opinion was pointed by Lafave [L5] that the wide beam specimens had greater slab participation than the conventional beam specimens. In addition, the specimens with deep rectangular columns had greater slab participation than those with square columns. Based on their test results, the simple effective tension slab width rules were developed and shown as below:

$$b_e = b_w + 2h_c \quad (6-2)$$

but not greater than:

$$b_e = 2b_c + h_c \quad (6-3)$$

It should be noted that these rules were developed for the exterior wide beam-column connection with floor slab and could be applied only when the transverse beam did not break down in torsion. **Equations 6-2** and **6-3** implied that the effective tension slab width was governed by the torsion stiffness or the torsion strength of the transverse beam. It also implied that the wide beam width to column width ratio should be limited to 2.

In the current tests and the tests carried out by Hatamoto *et al* [H4], the test specimens did not include floor slabs. However, based on the test results, the slab participation to the negative bending moment capacity could not be expected in these specimens because even the beam strength could not be developed in some cases. A concept of effective wide beam width was recommended by Hatamoto *et al* [H4], in which the effective wide beam width was defined as the width within which the beam longitudinal bar reached the yield strain. It is worth to note that this effective width will increase with the deformation level. In Hatamoto's Specimen WF-4 and Specimens IWB-1 and IWB-3 tested in this research, the effective beam width did not reach the actual beam width until the final loading cycle. It can be summarized that the effective wide beam width is controlled by the torsion strength of the transverse beams and, in addition, the effective beam width is sensitive to the beam to column width ratio rather than the column aspect ratio.

6.7 Summary

Based on the experimental and analytical results, some conclusions on the performance of the interior wide beam-column specimens can be summarized as follows:

- (1) Wide beam-column joints, when designed with suitable parameters, perform quite well in carrying lateral loads as generally they can obtain their strength and deformation capacity. The stiffness of wide beam-column subassemblages is relatively lower compared with that of ordinary connections, therefore, the large displacement under seismic forces should be considered in design.

- (2) Due to a larger section of the wide beam, the shear stresses in the beam transverse reinforcement were very low and as a result, the requirement of beam shear reinforcement can be relaxed.
- (3) In the wide-beam connections, lag in the development of the yield strain among the distributed beam longitudinal bars leads to a delay in the development of full-width hinges. Due to the shear lag, the deformation angles at which all beam bars reach yield strain increased. As the beam width ratio increased, the percentage of beam width that is effective decreased. Based on the current tests and previous tests conducted by Hatamoto *et al* [H4] , the effective beam width cannot reach the actual beam width when the beam to column width ratio is higher than 3.
- (4) The development of full-width hinges depends on the anchorage condition of the beam bars, especially for those bars are placed in the outside beam portions. Extra close stirrups should be provided to act as confinement to the beam portions which are outside of the column.
- (5) For the wide-beam specimens, the seismic performances are controlled by the torsion strength of transverse beam. As the torsion moment demand increase, the hysteretic loops show slip resulting in a decrease in the hysteresis energy dissipation. In some cases, the specimen will lose its load-carrying capacity after the transverse beam lost its torsion capacity. Therefore, the transverse beam is a critical issue for the wide-beam system design, which need carefully design and detailed.

Chapter 7

Test Results of Exterior Wide Beam-Column Joints

7.1 Introduction

Three full-scaled wide beam-column joint subassemblages, which were referred to as EWB-1 to EWB-3 were constructed and tested under simulated seismic loadings. All the specimens have the same dimensions and reinforcing details as those of the corresponding interior subassemblages with the exception that only one beam was framed into the connections. The main design parameters as well as the predicted specimen capacities were summarized in Chapter 4. This chapter presents the test results of these exterior wide beam-column specimens.

7.2 Test results of Specimen EWB-1

In this specimen, the ratio of the column to beam flexural strength in both loading directions was greater than 4.0 and as a result, plastic hinges were expected to form in the beam. The ratio of the column depth to beam bar diameter was 41/1 and the ratio of the beam depth to column bar diameter was 12/1. Due to the unsymmetrical reinforced details, the specimen presented different load-carrying capacities in different loading directions. In the positive loading direction (where the wide beam was bending in the negative moment), the theoretical capacity was 170.5 kN. On the other hand, when the beam was subjected to the positive bending moment, the theoretical capacity was 123.6 kN. The following sections report the test results of the specimen.

Figure 7-1 Observed cracking of Specimen EWB-1 at each selected storey drift level

7.2.1 General Observation

Cracking patterns were observed at different storey drift levels as shown in **Figure 7-1**. In the wide beam, flexural cracking initiated early at a storey drift ratio of 0.5% and in the subsequent cycles, the flexural cracking in both the positive loading and negative loading directions extended vertically and therefore, full-depth cracks were formed. At the same time, flexural cracks spread along more than three-fourths of the beam span length.

In the outside beam portions near the column, incline shear cracks opened and extended to the intersection lines between the column side and beam portions, which indicated a bond slip of the bars in the outside beam portions. At a storey drift ratio of 2.0%, a minor incline crack was observed in the column

Figure 7-2 Photograph of Specimen EWB-1 after testing

Up to the final loading stage, the beam flexural cracks and outside beam shear cracks increasingly developed; however, there was no shear cracking observed in the beam portions away from the column. **Figure 7-2** presents a photograph of Specimen EWB-1 after the test.

7.2.2 Hysteretic Behaviour

The storey shear force versus horizontal displacement relationship is shown in **Figure 7-3**. The specimen was finally loaded to a storey drift ratio of 4.0% in either loading direction. In the positive loading direction, the specimen did not reach its theoretical capacity and the maximum capacity was attained at a storey drift ratio of 1.0%, which was 150.6 kN or equalled to 88.3% of the theoretical capacity. In the following loading cycles, the strength of the specimen decreased gradually and at a storey drift ratio of 2.0%, this trend became significant. Up to the final loading cycle, the residual load-carrying capacity in this direction was 91 kN or 53% of the theoretical capacity.

Figure 7-3 Storey shear force versus horizontal displacement relationship for Specimen EWB-1

In the negative loading direction, the specimen obtained its theoretical capacity approximately at a storey drift ratio of 0.8%. The maximum capacity was obtained at a storey drift ratio of 0.9%, which was 132.4 kN exceeding the theoretical capacity based on the beam yield mechanism at 7%. This indicated that limited over-strength behaviour occurred in the negative loading direction. Up to a storey drift ratio of 2.0%, the specimen

retained its load-carrying capacity without obvious strength degradation. The strength in the negative loading direction started to decrease from a storey drift ratio of 2.5%, but the degradation was not significant.

The specimen experienced a decrease in the stiffness as the loading was applied. Due to the cracking of the concrete, yielding of the bars or slipping of the bars in the outside beam portions, the specimen carried lower loads during the repeated cycles. The hysteretic loops also showed a slight pinching behaviour with the pinching being detected at early storey drift levels, therefore, the stiffness dropped increasingly as a result.

7.2.3 Beam Behaviour

7.2.3.1 Strains in Beam Longitudinal Bars

The distribution of strain gauges on the beam bars is shown in **Figure 7-4**. The bars, which were gauged on the top layer were labeled from A to C. Strain gauges on the beam bottom bars were located at the corresponding locations to those on the top bars and were labeled from A' to C'. In this section and subsequent sections, only the strains in the beam tension zone are discussed.

Figure 7-4 Distribution of strain gauges on the beam bars of Specimen EWB-1

Figure 7-5 illustrates the strain profiles of the beam longitudinal bars that were anchored in the column core. Both the top and bottom bars started to yield at early loading stages in that

the top bars yielded at a storey drift ratio of 1.0% and the bottom bars yielded as early as a 1.5% storey drift ratio, which indicated that the beam suffered larger negative bending moment demands at the same loading levels.

Figure 7-6 illustrates the strain profiles of the beam longitudinal bars B and B'; which were placed in the outside beam portion near the column face. Strains in these bars did not increase evidently during the test with the maximum strain in the top bar being generated at the location near the central depth of the column, whereas, the maximum strains in the bottom bar being found near the column-beam intersection.

Figure 7-7 illustrates the strain profiles of the beam longitudinal bars C and C', which passing through the outside beam portion far away the column face. Strains in these bars were quite lower than those in the bars passing through the column core indicating an anchorage failure had occurred.

7.2.3.2 Strains in Beam Transverse Reinforcement

Two sets of beam transverse reinforcement, which were 50 mm and 350 mm away from the column face, were gauged (see **Figure 7-10**). **Figure 7-8** illustrates the strain profiles of the beam transverse reinforcement near the column face, where the recorded data showed that strains in the beam transverse reinforcement were quite low with the maximum value being about $500\mu\epsilon$. This indicated that the nominal shear stresses in the beam cross sections were very low as a result of the broad section area of the beam.

(a) Beam top bar-A

(b) Beam bottom bar-A'

Figure 7-5 Strain profiles of beam longitudinal bars-A and A' of Specimen EWB-1

(a) Beam top bar-B

(b) Beam bottom bar-B

Figure 7-6 Strain profiles of beam longitudinal bars-**B** and **B'** of Specimen EWB-1

(a) Beam top bar-C

(b) Beam bottom bar-C'

Figure 7-7 Strain profiles of beam longitudinal bars-C and C' of Specimen EWB-1

Figure 7-8 Strain profiles of beam transverse reinforcement of Specimen EWB-1

7.2.3.3 Beam Rotation Behaviour

Figure 7-9 illustrates the beam rotation under both the positive and negative loading directions. Beam rotation was measured based on the readings of pairs of LVDT distributed along the beam over a distance, which was equivalent to about $2h_b$, where h_b is the depth of the beam. As same as the rotation measurement applied to the interior test specimens, the rotation presented the average rotation at the beam sections located 70 mm and 195 mm away from the column face with an exception of one side beam being measured in the test. All the sections were located within the potential plastic hinges region.

A gradual increase in the beam rotation was observed during the test. Coinciding with the yielding of the beam longitudinal bars, the beam rotation increased significantly. At a storey drift ratio of 1%, the beam rotation reached 0.0035 rad. in the positive and 0.005 rad. in the negative loading directions, respectively. When the specimen was loaded into the final cycle, the maximum rotation estimated over the region nearest the column face reached 0.05 rad. in the positive loading direction, whereas when the specimen was

subjected to the negative loading, the maximum rotation increased significantly and reached at 0.04 rad. Therefore, the rotational ductility factors of the beam of Specimen EWB-1 were 14.3 in the positive and 8.0 in the negative loading directions, respectively. A large magnitude of the beam fixed-end rotation was expected to result in a greater contribution to the horizontal displacement which is shown in the following section.

Figure 7-9 Measured beam rotation of Specimen EWB-1

7.2.4 Column Behaviour

7.2.4.1 Strains in Column Longitudinal Bars

The distribution of strain gauges on the column bars is shown in **Figure 7-10**. During the test, only the corner bars were gauged and were labeled as D and E. The strain profiles of the column longitudinal bars are shown in **Figure 7-11**.

Figure 7-10 Distribution of strain gauges on the column bars of Specimen EWB-1

During all the loading stages, no yielding was observed in the column longitudinal bars indicating that the column remained essentially in the elastic range during the test. When the specimen was subjected to a positive loading, the upper column bar E attained the maximum strain up to the final loading cycle, but it only reached 40% of the yield strain. In the reverse loading direction, although the lower column bar E suffered a larger strain demand, the yield strain could still not be reached with the maximum value being $1350\mu\epsilon$ or 51.1% of the yield strain. As far as the loading cycle was completed, no tensional strains were found when the column was in the compression indicating that the column longitudinal bars did not lose their anchorage during the test. If comparing the strain profiles between the interior bar E and exterior bar D, it can be seen that at the same loading levels, the strain magnitude in the interior bars was higher than that in the exterior bars.

7.2.4.2 Strains in Column Transverse Reinforcement

Four sets of the column transverse reinforcement were gauged during the test. Some strain gauges had been damaged before the test was carried out and therefore, the data collection from the column transverse reinforcement strain recordings was not available. However, it still could be expected that the nominal shear stresses in the column cross sections were quite low and no yielding was attained in the column transverse reinforcement.

(a) Column bar-D

(b) Column bar-E

Figure 7-11 Strain profiles of column longitudinal bars-D and E of Specimen EWB-1

7.2.4.3 Column Rotation Behaviour

Up to the final loading stage, a gradual increase in the column rotation was observed. Due to the large flexural strength and stiffness, the magnitude of the column rotation was very limited at approximately equal to 10% of the beam rotation at the same loading cycles.

7.2.5 Joint Behaviour

From the observed cracking patterns, no joint shear failure was detected during the test. In the design and construction, joint transverse reinforcement was provided in this specimen to resist joint shear forces and to provide confinement to the joint core. Three sets of the joint transverse reinforcement were gauged to measure the strain variations.

Figure 7-12 Strain profiles of joint transverse reinforcement of Specimen EWB-1

Figure 7-12 illustrates the strain profiles of the joint transverse reinforcement. It was found that the middle-layer transverse reinforcement, which were approximately at the middle level of the joint, represented the largest strain demand with the maximum tension strain being about $1100\mu\epsilon$. Since the joint core was reinforced with the T16 deformed bars, the

yield strain of these bars was identical to the beam longitudinal bars with the same diameter, which was about $2500\mu\epsilon$, therefore the maximum strain in the joint transverse reinforcement was less than 50% of the yielding value.

7.2.6 Decomposition of Horizontal Displacement

Figure 7-13 illustrates the components of horizontal displacement at the peak of every loading run. At the early stages, the main sources which make up of the total horizontal displacement were uncounted factors. During a storey drift ratio of 1.0 to 1.5%, where the beam bars started to yield, the contribution due to the beam deformation increased significantly. In the remaining loading cycles, this contribution remained approximately in constant although the magnitude increased gradually until the final cycle. Up to a storey drift ratio of 4.0%, the beam fixed-end rotation and flexural deformation had contributed to the total horizontal displacement at percentages of 25.8% and 16.7%, respectively.

Figure 7-13 Percentage of deformation to the displacement of Specimen EWB-1

The contributions due to the column fixed-end rotation and flexural deformation were also increased with the applied loading, but these increases were limited. The maximum

percentages of the column flexural deformation and fixed-end rotation were only 2.3% and 2.5% of the total horizontal displacement, respectively. Until the final stage, an evident increase of the column deformation was not observed.

7.3 Test Results of Specimen EWB-2

The ratio of the column to beam flexural strength of Specimen EWB-2 reduced due to the different column orientation. In the positive loading direction (where the wide beam was bending in the negative moment), the theoretical capacity was 144.4 kN. Thus, the ratio of the column to beam flexural strength was 1.5. On the other hand, when the beam was subjected to the positive bending moment, the theoretical capacity was 103 kN, therefore, the ratio of the column to beam flexure strength was 0.68. As a result, plastic hinges were expected to form in the column and led to a strong beam-weak column configuration. The ratio of the column depth to beam bar diameter was 13.6/1 and the ratio of the beam depth to column bar diameter was 12/1. The following sections report the test results of the specimen.

7.3.1 General Observation

Cracking patterns were observed at different storey drift levels as shown in **Figure 7-14**. In the wide beam, flexural cracking initiated early at a storey drift ratio of 0.5%. In the loading at 1.5% storey drift ratio, column flexural cracks were observed in either the upper or bottom face, while beam flexural cracks extended vertically with the number of these cracks also increasing. At this stage, a diagonal crack was observed, which started from the right corner of the joint region and extended towards to the opposite corner of the joint.

As the specimen was loaded in a storey drift ratio of 3%, the beam flexural cracks spread over nearly the whole beam span length, whereas in the column; the crack quantity did not increase although the diagonal cracks extended into it. **Figure 7-15** presents a photograph of Specimen EWB-2 after the test.

Figure 7-14 Observed cracking of Specimen EWB-2 at each selected storey drift level

Figure 7-15 Photograph of Specimen EWB-2 after testing

7.3.2 Hysteretic Behaviour

The storey shear force versus horizontal displacement relationship is shown in **Figure 7-16**. In either the positive and negative loading direction, the specimen attained its theoretical capacity at a storey drift ratio of 0.8% and 0.6%, respectively. At the first cycle of a 2.0% storey drift ratio, the specimen reached its maximum capacity in the positive loading

direction, which was 185 kN, indicating that the specimen took an over-strength factor at 28% in this loading direction. In the subsequent positive loading cycles, the specimen retained its load-carrying capacity up until the final loading cycle without obvious strength degradation. In the negative loading direction, the specimen reached its maximum capacity at a storey drift ratio of 2.5%, which was 138.8 kN, indicating an over-strength factor at 34%. The specimen also retained its load-carrying capacity without strength degradation in this loading direction. However, due to the opening cracks and bond slip, the hysteretic loops pinched significantly and the specimen experienced a decrease in the stiffness as the loading was applied.

Figure 7-16 Storey shear force versus horizontal displacement relationship for Specimen EWB-2

7.3.3 Beam Behaviour

7.3.3.1 Strains in Beam Longitudinal Bars

The distribution of strain gauges on the beam bars is shown in **Figure 7-17**. The beam longitudinal bars were gauged at every one of the bars on the top and bottom layers. Strain

gauges on the top bars were labelled from A to C, strain gauges on the bottom bars were located at the same locations corresponding to those on the top bars and were labeled as A' to C'. Based on the measured strains, it is likely that the beam bars in Specimen EWB-2 presented with a similar strain behaviour and therefore in this section only the results of the bars A and A' which were placed in the centre of the beam are shown.

Figure 7-17 Distribution of strain gauges on the beam bars of Specimen EWB-2

Figure 7-18 illustrates the strain profiles of the beam top and bottom bars A and A'. The recorded data indicated a yielding initiation in the top bar A at a storey drift ratio of 1.5%, whereas in the bottom bar the yield strain was reached at a 1.0% storey drift ratio with the strain value significantly increasing with the deformation level. At higher loading levels, extensive damage was observed due to a large magnitude of deformation.

7.3.3.2 Strains in Beam Transverse Reinforcement

Also two sets of the beam transverse reinforcement, which were 50 mm and 350 mm away from the column face, were gauged. The recorded data showed that strains in the beam transverse reinforcement were quite low with the maximum value being about $530 \mu\epsilon$ indicating that the nominal shear stresses in the beam cross sections were very low as a result of the broad section area of the beam. **Figure 7-19** illustrates the strain profiles of the beam transverse reinforcement near the column face.

(a) Beam top bar-A

(b) Beam bottom bar-A'

Figure 7-18 Strain profiles of beam longitudinal bars-A and A' of Specimen EWB-2

Figure 7-19 Strain profiles of beam transverse reinforcement of Specimen EWB-2

7.3.3.3 Beam Rotation Behaviour

Figure 7-20 illustrates the beam rotation under both the positive and negative loading directions. A gradual increase in the beam rotation was observed during the test. In general, the wide beam showed a symmetrical rotation behaviour when the specimen was loaded in both the positive and negative loading directions. When the initial beam bar yielding occurred, the beam rotation at the section of the column face was 0.006rad. in the positive direction and 0.01rad. in the negative loading direction, respectively. In the positive loading direction, the maximum rotation estimated over the region nearest the column face was 0.026rad. leading to a rotation ductility factor of 4.3, while in the negative loading direction; the maximum rotation of the beam was 0.036rad. corresponding with a rotation ductility factor of 3.6.

Comparing this behaviour with that of Specimen EWB-1, it was noticed that the rotation magnitude of Specimen EWB-2 at the same loading levels was slightly lower. This might have been attributed to the relatively lower column stiffness and flexural strength ratio and

as a result, the deformation demand on the beam of Specimen EWB-2 was reduced and led to a decrease in the beam rotation.

Figure 7-20 Measured beam rotation of Specimen EWB-2

7.3.4 Column Behaviour

7.3.4.1 Strains in Column Longitudinal Bars

Figure 7-21 illustrates the distribution of strain gauges on the column bars. The strain profiles of the column longitudinal bars which were labeled as D and E are shown in **Figure 7-22**.

When the specimen was subjected a negative loading, tension strains were found slight increased with the applied loading in the upper column bar E, where the column was in the compression. This phenomenon indicated that an anchorage problem existed in the column bars even though the strain magnitude was limited. In the tension zone of the column, tension strains in the lower column bar E increased gradually as the loading increased,

however, the yield strain was not reached until the test completed with the maximum tension strain being about $1700\mu\epsilon$.

In the column bar D, desirable compression and tension strains were observed in the compression and tension zones. However, the strain level in this bar did not exceed the yielding level and it could be expected that the column always remained in the elastic range during the test.

Figure 7-21 Distribution of strain gauges on the column bars of Specimen EWB-2

7.3.4.2 Strains in Column Transverse Reinforcement

Four sets of the column transverse reinforcement were gauged during the test. The recorded data showed that the column transverse reinforcement suffered a low level strain demand and no yielding was found in the column transverse reinforcement. This phenomenon indicated that the nominal shear stresses in the column cross sections were quite low during the test.

(a) Column bar-D

(b) Column bar-E

Figure 7-22 Strain profiles of column longitudinal bars-D and E of Specimen EWB-2

7.3.4.3 Column Rotation Behaviour

Although the column stiffness of Specimen EWB-2 was much lower than that of Specimen EWB-1 or EWB-3, the column still performed with a good rotation response. Up to the final loading in the positive direction, the maximum rotation was reached at about 0.005rad. for the upper column and 0.003rad. for the lower column. In the negative loading direction, the rotation of the upper column dropped to 0.0015rad. The column deformation influence on the performance of the specimen can be detected from the components of the horizontal displacement and will be discussed in the later.

7.3.5 Joint Behaviour

Figure 7-23 illustrates the strain profiles of the joint transverse reinforcement at the middle layer. Comparing with Specimen EWB-1, the joint transverse reinforcement of Specimen EWB-2 represented slightly large strain demands, particularly at the middle level of the joint, which was approximately the centre of the diagonal concrete strut. After a diagonal crack initiated in the joint, strains in the joint transverse reinforcement increased dramatically with the maximum strain of 730 $\mu\epsilon$ being detected in the mid-layer transverse reinforcement. However, all the joint transverse reinforcement did not reach the yield strain indicating no shear problems in the joint region.

7.3.6 Decomposition of Horizontal Displacement

Figure 7-24 illustrates the components of horizontal displacement at the peak of every loading run. During the loading cycle of up to a 1.5% storey drift ratio, the main sources of the total horizontal displacement were some unexpected and unmeasured components, which included the joint shear deformation. During these stages, the unaccounted components contributed about 60% to the total horizontal displacement and the column and beam flexural deformation plus column and beam fixed-end rotation contributed to the remaining displacement.

In the loading to a storey drift ratio of 1.0%, the beam flexural deformation accounted for slightly more than 10% of the total horizontal displacement, and this was larger than the corresponding value of Specimen EWB-1 at this stage. Subsequent loading cycles resulted in an increment of the contribution of the beam flexure deformation. At a storey drift ratio of 2.5%, the contribution due to the beam flexure deformation was about 23% of the total horizontal displacement, while the beam fixed-end rotation represented a similar contribution as that of the beam flexure deformation, which increased gradually from 9% to 27% as the loading ran.

The contribution due to the column deformation could be ignored at an early stage, but in the subsequent loading cycles, the contributions due to the column flexure deformation and fixed-end rotation to the total horizontal displacement of Specimen EWB-2 were somewhat larger than those of Specimen EWB-1. After the specimen was loaded in a 1.5% storey drift ratio, the contributions of the column flexure deformation and fixed-end rotation increased significantly up until the final stage, where the column flexure deformation had contributed to 15% of the total horizontal displacement and the column fixed-end rotation was about 8%.

Figure 7-23 Strain profiles of joint transverse reinforcement of Specimen EWB-2

Figure 7-24 Percentage of deformation to the displacement of Specimen EWB-2

7.4 Test Results of Specimen EWB-3

Similar to Specimen EWB1, the column to beam flexural strength ratio of Specimen EWB-3 was greater than 4.0 resulting in a beam plastic hinge mechanism being able to be expected. The ratio of the column depth to beam bar diameter was 41/1 and the ratio of the beam depth to column bar diameter was 12/1. Based on the measured concrete strength of the specimen, the theoretical capacities of Specimen EWB-3 were 170 kN in positive and 123.6 kN in the negative loading directions. The following sections report the test results of the specimen.

Figure 7-25 Observed cracking of Specimen EWB-3 at each selected storey drift level

7.4.1 General Observation

Generally, Specimen EWB-3 presented similar cracking patterns to those of Specimen EWB-1. Cracking patterns were observed at different storey drift levels of Specimen EWB-3 as shown in **Figure 7-25**. In the wide beam, flexural cracking appeared early at a storey drift ratio of 0.5%. Flexural cracks due to both the positive and negative loadings extended vertically and spread over a certain portion of the beam length. There was no shear cracks detected indicating a low level shear stress demand in the beam.

Column flexural cracks were generated when a 2.0% storey drift ratio was reached. In the later cycles, minor extensions of these cracks appeared, while shear cracks in the outside beam portions near the column side were found at this stage, which indicated that some out-of-plane loading took place during the test. **Figure 7-26** presents a photograph of Specimen EWB-3 after the test.

Figure 7-26 Photograph of Specimen EWB-3 after testing

7.4.2 Hysteretic Behaviour

Figure 7-27 illustrates the storey shear force versus horizontal displacement relationship for Specimen EWB-3. The specimen did not reach its theoretical capacity in either loading direction. At a storey drift ratio of 1%, the specimen attained the maximum capacity of 160.9 kN, which was 94% of the theoretical capacity in the positive loading direction. Whereas, the maximum capacity was reached in the negative loading direction at a storey

drift ratio of 1.5%, which was 120.3 kN or equaled to 98% of the theoretical capacity. The specimen presented significant strength degradation after a storey drift ratio of 2.0%.

The specimen experienced a decrease in the stiffness as the loading was applied. Due to the cracking of the concrete and yielding of the bars, the specimen carried lower loads during the repeat cycles at each loading level. The hysteretic loops also indicated a pinching behaviour, which was detected at early drift levels, therefore the stiffness dropped increasingly as a result.

Figure 7-27 Storey shear force versus horizontal displacement relationship for Specimen EWB-3

7.4.3 Beam Behaviour

7.4.3.1 Strains in Beam Longitudinal Bars

Since Specimen EWB-3 had the same reinforcing details as Specimen EWB-1, therefore the strain gauge distributions were identical to those of Specimen EWB-1 and can be referred to in **Figure 7-4**. **Figure 7-28** illustrates the strain profiles of the beam longitudinal

bars that were anchored in the column core. **Figures 7-29 and 7-30** illustrate the strain profiles of the beam longitudinal bars that were placed in the outside beam portion

In general, strains in the beam longitudinal bars of Specimen EWB-3 were very similar to those of Specimen EWB-1. All of the beam top and bottom bars anchored in the column core were yielded at or before a storey drift ratio of 1.5%. The yielding was found at the position nearest the section where the maximum bending moment was encountered during the test.

The bars placed in the outside beam portion did not reach the yield strain with the maximum strain being generated at the location near the center depth of the column during the test. This indicated that a lag in the yielding occurred among the bars passing through the column and outside beam portion so that an anchorage problem could be expected in the beam bars within the outside beam portions.

7.4.3.2 Strains in Beam Transverse Reinforcement

Strains in the beam transverse reinforcement of Specimen EWB-3 showed very similar strain behaviour as that of Specimen EWB-1. The recorded data showed that strains in the beam transverse reinforcement were quite low with the maximum value being about $450 \mu\epsilon$ indicating that the nominal shear stresses in the beam cross sections were very low as a result of the broad section area of the beam. **Figure 7-31** illustrates the strain profiles of the beam hoop near the column face.

7.4.3.3 Beam Rotation Behaviour

Figure 7-32 illustrates the beam rotation under both the positive and negative loading directions. A gradual increase in the beam rotation was observed during the test. In the positive loading direction, the maximum rotation estimated over the region nearest the column face was 0.03rad., whereas when the specimen was subjected to a negative loading, the maximum rotation of the beam was 0.035rad. For those sections away from the column face, the rotation values were quite low at no more than 0.01rad. At the final loading cycle,

the rotation ductility factor was 6.7 in the positive and 10 in the negative loading directions, respectively.

Figure 7-28 Strain profiles of beam longitudinal bars-A and A' of Specimen EWB-3

(a) Beam top bar-B

(b) Beam bottom bar-B'

Figure 7-29 Strain profiles of beam longitudinal bars-**B** and **B'** of Specimen EWB-3

(a) Beam top bar-C

(b) Beam bottom bar-C'

Figure 7-30 Strain profiles of beam longitudinal bars-C and C' of Specimen EWB-3

Figure 7-31 Strain profiles of beam transverse reinforcement of Specimen EWB-3

Figure 7-32 Measured beam rotation of Specimen EWB-3

7.4.4 Column Behaviour

7.4.4.1 Strains in Column Longitudinal Bars

Figure 7-33 illustrates the strain profiles of the column longitudinal bars D and E which had the similar behaviour to that of Specimen EWB-1. As was observed, yielding was not observed along the column bars as far as the final loading cycle was completed. The desirable compression and tension strains were observed in the compression and tension zones of the column indicating that the column longitudinal bars were well anchored during the test and column remained essentially in the elastic range during the test. The contribution due to the column deformation can be found in **Figure 7-35**.

7.4.4.2 Strains in Column Transverse Reinforcement

Four sets of the column transverse reinforcement were gauged during the test. The recorded data showed that the column transverse reinforcement suffered a low level strain demand and no yielding was found in them. This phenomenon indicated that the nominal shear stresses in the column cross sections were quite low during the test.

7.4.4.3 Column Rotation Behaviour

The column rotation of Specimen EWB-3 was similar to that of Specimen EWB-1. Up to the final loading stage, the column rotation gradually increased as the lateral loading was applied. In general, column rotation was on a very small scale compared to the wide beam rotation response. This might be mainly due to the relatively large stiffness of the column, while it also showed that the behaviour of the column was still in its elastic range.

(a) Column bar-D

(b) Column bar-E

Figure 7-33 Strain profiles of column longitudinal bars **D** and **E** of Specimen EWB-3

7.4.5 Joint Behaviour

Figure 7-34 illustrates the strain profiles of the joint transverse reinforcement. Specimen EWB-3 had identical joint transverse reinforcement which contained three sets of the T16 deformed hoops. There was no yield strain was found in the joint transverse reinforcement indicating that the joint was not undergoing a shear problem and perhaps the joint concrete was probably able to carry shear forces via the diagonal concrete compression strut mechanism. After the opening of joint cracks, the joint transverse reinforcement acted to confine the cracked concrete strut therefore the joint was able to carry the shear forces via the diagonal concrete compression strut mechanism.

Figure 7-34 Strain profiles of joint transverse reinforcement of Specimen EWB-3

7.4.6 Decomposition of Horizontal Displacement

Figure 7-35 illustrates the components of horizontal displacement at the peak of every loading run. The beam displacement due to the flexure deformation and fixed-end rotation increased gradually during the test. The beam fixed-end rotation accounted for approximately 2% to 30%, while the beam flexure deformation accounted for approximately 5% to 41% to the total horizontal displacement. The contribution due to the

beam deformation increased up until the final test stage and the trend of them to the total horizontal displacement was slightly larger than that obtained from Specimen EWB-1.

As shown in **Figure 7-35**, the contribution of the column fixed-end rotation slightly increased during the test and the column flexure contribution to the total horizontal displacement was similar to that of the column fixed-end rotation.

Figure 7-35 Percentage of deformation to the displacement of Specimen EWB-3

7.5 Three Dimensional Finite Element Evaluations on Exterior Wide Beam-Column Joint Response

7.5.1 Introduction

Once again, a finite element evaluation was conducted to provide additional insight into the exterior joint behaviour with the finite element models being constructed to match the as-built dimensions and material properties. **Figure 7-36** illustrates the finite element models and boundary conditions.

Figure 7-36 Analytical models and boundary conditions

7.5.2 Verification of Experimental and Analytical Results

7.5.2.1 Comparison of Global Behaviour

Global behaviour is compared in terms of the comparison of hysteretic behaviour. **Figures 7-37 and 7-38** illustrate the hysteretic responses obtained through finite element analyses and experimental investigations. These figures indicate that the overall behaviour between the analytical and experimental results is basically in good agreement.

Figure 7-37 illustrates the comparison of hysteretic behaviour between Specimen EWB-1 and Model EWB-1. In the analysis, the theoretical capacity was reached as early as at a storey drift ratio of 0.8% in the negative loading direction, whereas, in the positive loading direction, the model did not reach the theoretical capacity. These phenomena were same as the experimental investigation. In addition, in either loading direction, the maximum capacity was gained as the model was loaded into a storey drift ratio of 1.0% to 1.5%, which was 152 kN and 158 kN in the negative and positive loading directions, respectively. Therefore, the maximum capacities of Model EWB-1 were higher than the measured values in Specimen EWB-1 at 15% and 5%, respectively, which resulted in higher stiffness at early stages. After the maximum capacity was reached, the strength and stiffness

degradation were obvious in the positive loading direction. In all the analytical loops, the pinching behaviour, which had been observed during the test, could not be captured by the finite element analysis approach.

Figure 7-38 indicates that the hysteretic behaviour of Model EWB-2 was underestimated. Until the final loading cycle, the load-carrying capacities of Model EWB-2 were lower than the experimental results. The maximum capacities were obtained by Model EWB-2 at a storey drift ratio of 2.5%, which were 169 kN in the positive and 125 kN in the negative directions, respectively. According to the experimental results, Specimen EWB-2 gained its load-carrying capacities at 179 kN in the positive and 138.8 kN in the negative directions, which were 5% and 11% higher than Model EWB-2 at this stage, respectively.

Figure 7-37 Comparison of hysteretic behaviour between Specimen IWB-1 and Model IWB-1

Figure 7-38 Comparison of hysteretic behaviour between Specimen IWB-2 and Model IWB-2

7.5.2.2 Comparison of Strains in Beam Longitudinal Bars

The strain behaviour of the analytical models is shown in **Figures 7-39 to 7-44**. The bars' labels are identical to those defined in the tests and can be referred to in **Figures 7-4, 7-17**. Since the measured strains in the beam longitudinal bars in Specimens EWB-1 and EWB-3 were quite similar, therefore only the analytical results of Model EWB-1 were presented in this section.

In Model EWB-1, the calculated strains in the beam longitudinal bars indicated that the bars anchored in the column were subjected to greater strain demands than those anchored in the transverse beam, which had been proved in the experimental investigations. From finite element analysis, the maximum strain was obtained at the intersection of the column to beam and the calculated strain magnitude in the internal bars was much higher than the measured value with yielding penetration being observed in the column core.

For the outmost bars, due to the poor anchorage condition between the bars and surrounding concrete, the beam bars could not transfer the tension forces into the column via torsion moments within the transverse beam.

In Model EWB-2, strains in the beam top as well as bottom longitudinal bars were quite close. The calculated yield strain was obtained before or at a storey drift ratio of 1.5% with the strain values in the bottom bars being higher than those in the top bars, which were also found in the test (sees **Figures 7-42 to 7-44**).

7.5.2.3 Comparison of Strains in Column Longitudinal Bars

The calculated strains in the column longitudinal bars are shown in **Figures 7-45 and 7-46**. Tension strains in the column bars increased evidently as the models were loaded from a storey drift ratio of 0.5% to 1.0%. In Model EWB-2, strains in the column bars increased progressively with the applied loading. Due to the lower flexural strength and stiffness, the column bars in Model EWB-2 was subjected to a larger magnitude of tension strain. In addition, tension strains were found in the column compression zone, which indicated an anchorage problem. By contrast, in Model EWB-1, strains did not increase considerably in the remaining loading cycles. In any cases, the yield strain was not reached in the column bars whose behaviour remained essentially in the elastic range.

(a) Beam top bar-A

(b) Beam bottom bar-A'

Figure 7-39 Strain profiles of beam longitudinal bars-A and A' of Model EWB-1

(a) Beam top bar-B

(b) Beam bottom bar-B'

Figure 7-40 Strain profiles of beam longitudinal bars **B** and **B'** of Model EWB-1

(a) Beam top bar-C

(b) Beam bottom bar-C'

Figure 7-41 Strain profiles of beam longitudinal bars-C and C' of Model EWB-1

(a) Beam top bar-A

(b) Beam bottom bar-A'

Figure 7-42 Strain profiles of beam longitudinal bars-A and A' of Model EWB-2

(a) Beam top bar-B

(b) Beam bottom bar-B'

Figure 7-43 Strain profiles of beam longitudinal bars-**B** and **B'** of Model EWB-2

(a) Beam top bar-C

(b) Beam bottom bar-C'

Figure 7-44 Strain profiles of beam longitudinal bars-C and C' of Model EWB-2

(a) Column bar-D

(b) Column bar-E

Figure 7-45 Strain profiles of column longitudinal bars-D and E of Model EWB-1

(a) Column bar-D

(b) Column bar-E

Figure 7-46 Strain profiles of column longitudinal bars **D** and **E** of Model EWB-2

7.5.2.4 Evaluation of Effective Beam Width

Strain variations in the beam longitudinal bars at the sections adjacent to the column faces of Models EWB-1 and EWB-2 are shown in **Figures 7-47** and **7-48**. The effective beam width of these exterior models can be evaluated through these figures.

As mentioned earlier, in Model EWB-1, the beam bars anchored in the column core reached the yield strain as early as a storey drift ratio of 1.0%, but for the bars anchored in the transverse beam did not reach the yield strain until the end of the analysis. This phenomenon can be clearly found in **Figure 7-47**. At a storey drift ratio of 1.0%, only the beam bars anchored in the column core obtained the yield strain, therefore, the effective beam width at this moment equals to the column width. Within the portions which extended to 100 mm from the both column faces, the beam longitudinal bars also reached the yield strain between storey drift ratios of 1.0% to 1.5%. Therefore, the effective beam width of Model EWB-1 can be taken as the column width plus 100 mm at each side of the column face.

Figure 7-47 Strain variations in the beam top bars of Model EWB-1

Figure 7-48 Strain variations in the beam top bars of Model EWB-2

In Model EWB-2, the beam longitudinal bars were fully yielded between storey drift ratios of 1.5% to 2.0%, therefore, the effective beam width is the actual width of the beam.

7.6 Discussion of Experimental and Analytical Results of Exterior Wide Beam-Column Joints

7.6.1 Load-Carrying Capacity and Hysteretic Response

The storey shear force versus horizontal displacement relationships obtained through either experimental investigations as well as finite element analyses show that the exterior wide beam-column Specimens EWB-1 and EWB-3 did not reach their theoretical capacities in the positive loading direction, but gained them in the negative loading direction and presented with a limited over-strength behaviour in Specimen EWB-1. However, an obvious strength enhancement was found in Specimen EWB-2. In these specimens, the yield displacement was reached between storey drift ratios of 1.0% to 1.5%, at which moment the beam bars reached the yield strain and as a result, all the test specimens could obtain a ductility factor of 2.0 without significant strength degradation.

As can be seen from the experimental and analytical results, only the beam longitudinal bars within the effective beam width yielded. As the deformation level past the yielding level, some of the beam longitudinal bars, particularly those anchored in the column, achieved strain values well in excess of the yielding and at larger deformation levels, strain hardening in these bars became more evident. Due to the bars anchored in the transverse beam did not develop the tension or compression yield, therefore; the over-strength behaviour was not significant in Specimens EWB-1 and EWB-3. This can also be taken as the reason why these test specimens could not reach their theoretical capacity.

Comparing the test results of Specimen EWB-2 with those of Specimen EWB-1 or EWB-3, it is noticed that a lower beam to column width ratio in Specimen EWB-1 or EWB-3 did not result in a noticeable improvement of the response. In contrast, Specimen EWB-2 presented a better performance than other specimens did. In Specimen EWB-2, the theoretical capacities were attained between storey drift ratio of 0.6% and 0.8% in both loading directions. Due to the different anchorage detail of the beam bars in the joint core, the bond slip was not found in the test in this specimen.

All of the exterior specimens met the ACI-ASCE 352 [A3] recommendation on flexural strength ratio leading to a “strong column-weak beam” configuration. Columns in Specimens EWB-1 and EWB-3 were much stronger than the beams; as a result it led to a low strain level in the column bars as well as an elastic response of the column during the test. From strain profiles of the column longitudinal bars, it can be seen that column bars in all the specimens did not reach the yield strain until the end of the test.

7.6.2 Comparison of Stiffness Degradation

Figure 7-49 illustrates the stiffness degradation for Specimens EWB-1 and EWB-2 based on the measured results. Due to the unsymmetrical load-carrying capacities in different loading directions, the specimens present different stiffness behaviour. As shown in **Figure 7-49**, the stiffness degradation due to the positive loading is plotted with dash lines while solid lines describe the stiffness degradation due to the negative loading.

Figure 7-49 Stiffness degradation in the exterior wide beam-column specimens

In general, the stiffness of all these specimens decreased dramatically when they were loaded from a storey drift ratio of 0.5% to 1.5% and beyond the storey drift ratio of 1.5%, the stiffness decreased slowly as the loading ran.

In Specimen EWB-1, the initial positive stiffness was 9 kN/mm, which was 23% higher than the initial negative stiffness. However, due to significant strength degradation in this loading direction, the positive stiffness decreased rapidly from a storey drift ratio of 0.5% to 1.5 % with the residual stiffness being about 3.5 kN/mm, which was 39% of the initial stiffness. But after a storey drift ratio of 1.5% up until to the end of the test, this rapid degradation trend became slowly and at the end of the test, the residual stiffness in the positive direction was 0.83 kN/mm, which was slight lower than the stiffness in the negative direction of 1.0 kN/mm. In the negative direction, the specimen also experienced strength degradation during the test. The initial negative stiffness of 7.3 kN/mm decreased gradually from a storey drift ratio of 0.5% to 1.5 % and at a drift ratio of 1.5%, the residual stiffness was 3.1 kN/mm, which equaled to 42.5% of the initial value. Beyond a storey drift

ratio of 1.5% up until to the end of the test, stiffness degradation trend curves in either loading directions were very close.

In Specimen EWB-2, the specimen carried its load-carrying capacities in both loading directions without obvious strength degradation, the stiffness did not decrease rapidly with the applied loading and as a result the positive stiffness was always higher than the negative ones.

Comparing the stiffness behaviour of Specimen EWB-1 to that of Specimen EWB-2, it is found that Specimen EWB-1 possessed the higher stiffness in an early stage. Since the column of Specimen EWB-2 was more flexible than that of Specimen EWB-1, Specimen EWB-2 represented with the lower initial stiffness. However, due to the strength degradation in Specimen EWB-1 and considerable strain hardening in Specimen EWB-2, stiffness of Specimen EWB-2 finally exceeded that of Specimen EWB-1, which indicated that Specimen EWB-2 performed well during the test although the theoretical capacity and flexural strength ratio were lower.

The stiffness magnitude of Specimen EWB-3 is similar to Specimen EWB-1, so it is not shown in this section.

7.6.3 Anchorage Conditions for the Beam and Column Longitudinal Bars

7.6.3.1 Beam Longitudinal Bars Anchored in the Column and Transverse Beams

In Specimens EWB-1 and EWB-3, the beam bars' anchorage ratio and the column depth to beam bar diameter ratio were identical to those of the interior specimens and therefore, a sufficient anchorage for the beam longitudinal bars anchored in the columns could be expected and has been validated by the experimental and analytical results. An additional anchorage was provided by the bend hook of the beam longitudinal bars which were anchored in the transverse beam. Therefore, the anchorage conditions in the outside beam portions were improved compared with that in the interior specimens. The observed strain

behaviour of the beam longitudinal bars in Specimens EWB-1 and EWB-3 indicated however, the full-width plastic hinges in the wide beam could not be developed.

In the exterior wide beam-column connections, the torsion strength and stiffness of the transverse beam are important to the overall performance. In Specimens EWB-1 and EWB-3, the significant strength and stiffness degradation were observed in the positive loading direction which might be attributed to the torsional failure of the transverse beam. Although the torsion cracks in the outside beam portions did not penetrate into the full transverse beam region, however, it still resulted in an additional rotation of the wide beam and thus the significant pinching in the hysteretic response and the deterioration in the hysteretic energy absorption capacity could not be avoided. The transverse reinforcement in the outside beam portions was expected to ensure the beam longitudinal bars developing the sufficient anchorage and to increase the torsional stiffness of the outside beam portions after cracking.

7.6.3.2 Column Bar Anchorage

As aforementioned, an anchorage problem was detected in Specimen EWB-2, where the tension strains were found in the column compression zone. However, the strain level in the column bars was quite low and therefore the extensive anchorage failure was not observed during the test.

The column exterior and interior bars suffered from different anchorage conditions. In the column interior bars, additional confinement was provided by the beam portions, but this could not be expected in the column exterior bars, in which, the column exterior bars were only confined by the column transverse reinforcement. Insufficient confinement and concrete cover spalling will result in the anchorage failure of the column bars.

7.6.4 Joint Shear

During the test, there were neither strains nor cracking patterns indicate that the joints were undergoing shear problems. Finite element analysis clearly shows that most of the joint

shear force was carried by the diagonal concrete compression strut mechanism during loading processed and after the joint shear cracking was generated, the transverse reinforcement was required to carry most portion of the shear force. However, in the tests as well as in the finite element analyses, strains in the joint transverse reinforcement retained a lower value and a large joint area was effective in reducing the nominal joint shear stresses. In addition, the column longitudinal bars were also helpful in resisting the shear forces.

Figure 7-50 Diagonal concrete struts in the joint core via finite element analysis

In an exterior beam-column connection, the imposed joint shear force was lower than that in the interior connections. For the all specimens, the maximum nominal horizontal shear stress in the joint core was lower than the designed limit. Although the current test results show that the strain values of the joint hoops were quite low, but the joint hoops are still important in the design and construction of exterior beam-column joints. The role of joint hoops is not only to participate in the joint shear resisting but also to provide joint confinement and prevent longitudinal bars buckling as well as straightening of beam bar end hooks.

7.6.5 Effective Wide Beam Width of Exterior Beam-Column Joints

The test on the exterior wide beam-column connections associated with the previous discussions about interior wide beam-column connections, the effective wide beam width was found to be increased with the applied loading. In the current test specimens with a beam width to column width ratio equals to 3, the maximum effective beam width was approximately 500 mm. This might be attributed to an insufficient torsion moment transfer mechanism. In the design and assessment of wide beam structures, this must be taken into

account to avoid overestimating the capacity of the structure as well as the individual members.

7.7 Summary

Based on the experimental and analytical results of the exterior wide beam-column specimens, some conclusions can be summarized as:

- (1) All the three exterior specimens started to yield between storey drift ratios of 1% to 1.5%. Specimen EWB-2 gained and exceeded its theoretical capacities in both loading directions and demonstrated over-strength behaviour. However, Specimens EWB-1 and EWB-3 only reached their theoretical capacities in the negative loading direction and in the positive loading direction; significant strength degradation was found.
- (2) All the specimens were tested to a storey drift ratio of 4% in either loading direction and experienced displacement ductility factors of approximately 3.5 to 4. In Specimens EWB-1 and EWB-3, the obtained positive displacement ductility without the loss in the load-carrying capacity was about 2.
- (3) Specimen EWB-2 performed well, even though the bar diameter to anchorage depth ratio did not satisfy the design recommendations which implies that the performance of a wide beam-column connection can be improved when the beam bar anchorage ratio is increased.
- (4) The wide beam shear force is not significant to the overall performance; as a result, the requirement of beam transverse reinforcement can be relaxed.
- (5) The inadequate performance of Specimens EWB-1 and EWB-3 is mainly due to the cracking of the transverse beam, which indicated that the seismic performance of the exterior beam-column connections is sensitive to the torsion strength of the transverse beams

Chapter 8

Parametric Investigations on Lightly Reinforced Beam-Column Joints and Wide-Beam-Column Joints

8.1 Introduction

It appears from the preceding chapters that, in a lightly reinforced moment-resisting frame the beam hinge mechanism might not arise. On the other hand, in a wide beam moment-resisting frame structure, the joint shear distortion was not significant. Thus, the behaviour of these types of joints subjected to reversed cyclic loadings may be different to that of the seismic designed reinforced concrete beam-column joints. In order to provide detailed description of the shear strength and behaviour of these kinds of joints, it is necessary to carry out an extensive parametric study on the variables that affect the behaviour of beam-column joints, which are designed without seismic provisions, under earthquake actions.

This chapter conducts a parametric study by using the results from experiment and non-linear finite element analysis with aim of establishing the relationship between variables and joint shear resistance under lateral loads. The most important outcome of the research described in this chapter is devoted to improve the understanding of the influences of variables on the behaviour of beam-column joints designed without seismic provisions.

8.2 Review of Previous Parametric Studies and Suggested Design Equations on Reinforced Concrete Beam-Column Joints

The induced maximum shear stress within a joint is one of the key parameters when assessing the performance of a beam-column joint. The mechanism of a beam-column joint under reversed cyclic loadings is extremely complex because the joint behaviour is governed by several factors associated with their interaction, therefore, the variables for the parametric study must be chosen according to design procedures and the uncertainties of being adopted by current code provisions.

8.2.1 Present Design Code Equations

The procedure for calculating joint shear forces has been described in Chapter 2 (**Equations 2-3 to 2-7**). Many design codes, such as ACI-ASCE 352 [A3], NZS 3101 [C1] and AIJ [A9], specify design provisions to limit the nominal shear stresses of beam-column joints. The maximum nominal joint shear stress is normally limited in relation to the concrete compressive strength such as the ACI-ASCE 352 [A3] recommendation, in which the joint shear stresses are limited by a square root of the concrete compressive strength multiplied by a factor. While design codes NZS 3101 [C1] and AIJ [A9] are described in terms of the concrete compressive strength multiplying a different factor according to the joint type.

Design of a reinforced concrete structure or element is in an effort to control failure mechanisms therefore, the values and influences of various parameters must be well understood. Nevertheless, the variable influences are not reflected in the design code equations directly.

8.2.2 Proposed Equations by Other People

For the beam-column joints, the key parameters may include the ratio of column to beam flexural strength, column axial load level, beam and column longitudinal reinforcing ratio, joint confinement, bond condition and the presence of a transverse beam and slab. By carrying out a database study and analytical investigations, Pantazopoulou *et al.* [P14], Vollum *et al* [V2] and Bakir *et al* [B5] proposed the equations to reflect the influence of variables on the joint shear strength for both interior and exterior connections. Their equations are presented as follows:

Bakir's Equation [B5]:

$$V_j = \frac{0.71\beta\gamma \left(100 \times \frac{A_{sb}}{b_b d_b}\right)^{0.4289} \left(\frac{b_c + b_b}{2}\right) h_c \sqrt{f_c}}{\left(\frac{h_b}{h_c}\right)^{0.61}} + \alpha A_{sj} f_y \quad (8-1)$$

In the above equation, the last term reflects the influence of joint transverse reinforcement. If ignoring the joint transverse reinforcement's influence, below equation is suggested:

$$V_j = \frac{0.71\beta\gamma \left(100 \times \frac{A_{sb}}{b_b d_b}\right)^{0.4289} \left(\frac{b_c + b_b}{2}\right) h_c \sqrt{f_c}}{\left(\frac{h_b}{h_c}\right)^{0.61}} \quad (8-2)$$

In these two equations, variable influences such as beam longitudinal reinforcing ratio, joint aspect ratio, concrete compressive strength and joint transverse reinforcing ratio are highlighted. However, the Bakir's equation did not reflect the column axial load effect.

Vollum's Equation[V2]:

$$V_c = 0.624\beta(1 + 0.555(2 - \frac{h_b}{h_c}))b_{eff}h_c\sqrt{f_c} \quad (8-3)$$

Vollum used this equation to describe the joint shear strength of exterior joints without stirrups. The variable β reflects the difference of beam reinforcing details in the exterior joint such as U bar or L bar and the constant “2” reflects joint aspect ratio. Author suggested that the joint shear strength should be limited to:

$$V_c < 0.624\beta(1 + 0.555(2 - \frac{h_b}{h_c}))b_{eff}h_c\sqrt{f'_c} < 1.33\sqrt{f'_c}b_{eff}h_c \quad (8-4)$$

Pantazopoulou and Bonacci's Equation:

By using the average stresses for equilibrium of a beam-column joint, Pantazopoulou and Bonacci [P14] established a series of principles of the joint shear mechanics, in which the shear stresses in a joint are dependent on the concrete principal tensile strains, which can be expressed as below:

$$\varepsilon_1 = \frac{1}{E_s \left(1 - \left(\frac{h_b}{h_c} \right)^2 \right)} \left(\tau_{av} \frac{h_b}{h_c} \left(\frac{1}{\frac{A_{sb}}{b_{eff}h_b}\mu + \frac{A_{sje}}{b_{eff}h_b}} - \frac{1}{\frac{A_{scol}}{b_{eff}h_c}} \right) + \frac{N \left(\frac{h_b}{h_c} \right)^2}{A_{scol}} \right) \quad (8-5)$$

Evidently, in this equation, increasing of the joint aspect ratio, column longitudinal reinforcing ratio and column axial load will increase the principle tensile strains of concrete; whereas they are decreased by increasing beam longitudinal reinforcing ratio and stirrup ratio. Meanwhile the joint shear strength increases as the beam longitudinal reinforcing ratio, the joint transverse reinforcing ratio, the column axial load and the column longitudinal reinforcing ratio increased. Since the increase in the concrete principal tensile strains will result in a decrease in the joint shear strength, the increase in the joint shear strength will be offset.

The author also derived general expressions for limiting joint shear resistance [P14]:

(1) Steel yield in two directions:

$$v_n = \sqrt{(\rho_t f_{yt} - n_h)(\rho_l f_{yl} - n_v)} \quad (8-6)$$

(2) Steel yield in horizontal direction+crushing:

$$v_n = \sqrt{(\rho_t f_{yt} - n_h)(\lambda f'_c - \rho_l f_{yt} + n_h)} \quad (8-7)$$

(3) Steel yield in vertical direction+crushing:

$$v_n = \sqrt{(\rho_l f_{yl} - n_v)(\lambda f'_c - \rho_t f_{yt} + n_v)} \quad (8-8)$$

In the above equations, v_n is the limiting joint shear stress; ρ_t and f_{yt} are the ratio and yield stress of horizontal reinforcement; ρ_l and f_{yl} are the ratio and yield stress of vertical reinforcement; n_h and n_v are the horizontal beam's axial stress and vertical column's axial stress respectively; f'_c is the concrete compressive strength and λ is a coefficient dependent on concrete constitutive model that reflects the influence of confinement and strain softening [P14].

Obviously axial stresses both in the beam and column, along with the quantity and strength of joint reinforcement, play an active role in determining the shear capacity of connections, while the effectiveness of joint hoops in providing confinement is implicitly included in defining the maximum tolerable compressive stress in the concrete core [P14].

8.2.3 Critique of Previously Suggested Design Equation

It is noted that the above designed equations were derived either from regression analysis of the experimental data or from numerical analysis of the joint shear mechanism. Both Bakir's and Vollum's equations were developed from the parametric study on the exterior beam-column joints. Although the variable influences were reflected in these equations, they might not be applicable for an interior beam-column joint. Pantazopoulou's and Bonacci's equations provide a general limiting for the joint shear stresses regardless of

the joint types, but they were developed with an idealized joint shear mechanism. It was pointed out that these researches were not intended to be either a nomination of new design equations or an affirmation of existing recommendations. Rather, they were presented to show how the equations provide devices to investigate the variable influences on the joint behaviour [P14].

It can be summarized from the previous studies that the parameter influences must be evaluated in association with failure modes. Since the existing problems for limited or non-ductile designed beam-column joints attributed to a low column longitudinal reinforcing ratio, a “weak column-strong beam” configuration, no shear reinforcement provided within the joint and insufficient bond conditions, the current design code’s limits and previously established equations might not be directly applicable on these kinds of beam-column joints. Therefore, the parameters selected for this study are the ratio of beam longitudinal reinforcement, different reinforcing details, column axial load level, and the influence of presence of a transverse beam and slab. Since the joint transverse reinforcement is not provided in non-seismic designed beam-column joints, the influence of joint transverse reinforcement will not be considered in the current study.

8.3 Assembly of Experimental Database

8.3.1 Experimental Results of Conventional Beam-Column Joints

Altogether 33 interior reinforced beam-column joint assemblies without slab (except for three specimens tested by the author) were summarized with attention to the lightly reinforced beam-column joints without joint transverse reinforcement. These tests were conducted in New Zealand, Italy and Singapore recently. All specimens were designed and constructed with typical reinforcing details, which were used in the gravity-load-designed reinforced concrete frames. All specimens were subjected to reversed cyclic loads to simulate seismic loadings with some of these specimens being loaded by vertical

displacing at the end of the beam and others being loaded by lateral displacing at the top of the column.

Normal concrete with compressive strength ranging from 30 MPa to 53 MPa was used in all the test specimens, with an exception of Specimen R2 tested by Hakuto *et al* [H1], for which the concrete compressive strength of 61 MPa was reported. A variety of steel types was used for the reinforcement with the yield strength ranging from 306 MPa to 510 MPa. **Table 8-1** tabulates the design parameters of these specimens with the maximum joint shear stress of the specimens ranging from $0.23 \sqrt{f'_c}$ to $1.51 \sqrt{f'_c}$. For the convenience of comparison, other information including material properties, column axial load level, column to beam flexural strength ratio, beam longitudinal reinforcing ratio, ratios of beam depth to column bar diameter and column depth to beam bar diameter are also provided in **Table 8-1**.

8.3.2 Experimental Results of Wide Beam-Column Joints

In a wide beam structural system, the additional parameters, such as the beam width to column width ratio, the beam longitudinal bar anchorage ratio, the confinement in the outside beam portions and the column section aspect ratio are all very important to the behaviour of wide beam-column connections. These parameters have been mentioned and investigated in the earlier experiments conducted by Wight *et al*. [G2, Q1, and L5]. However, the parameter influences could not be fully investigated through the experimental studies because each of these parameters even when independent is not entirely understood. Moreover, many of these parameters are actually interrelated. Thus, for the purpose of convenience, they were considered one at a time during the particular test.

Although the shortcomings of the previous investigations exist, the experimental information is worthy of consideration because of the scarcity of test data in the literature

on wide beam-column connections. Based on the author's knowledge, so far there are only six tests having been reported, which contained fifteen interior and eight exterior wide beam-column connections. Most of these experiments were conducted in the 1990s. **Tables 8-2** and **8-3** tabulate the design parameters of these test specimens.

Table 8-1 Parameters of lightly reinforced beam-column joints

Reference	Specimen	f'_c (MPa)	f_y (MPa)	$N / A_g f'_c$	M_r	ρ_{top} (%)	ρ_{bot} (%)	h_c / d_b	h_b / d_c	BI	$v_{jh}(\sqrt{f'_c})$
Blaikie [B4] (1998)		29.6	352	0	0.93	1.45	1.09	20.8	19.5	1.56	1.0
Liu, A. <i>et al.</i> [L2] (1998)	Unit 1	43.8	321	0	0.57	1.36	0.68	12.5	19.2	1.94	0.5
	Unit 2	48.9	321	0.12	1.05	1.36	0.68	12.5	19.2	1.84	0.79
Pessiki <i>et al.</i> [P8]	1	24.1	413.7	0.39	0.54	1.19	0.59	14.2	18.9	2.97	0.9
	2	24.1	413.7	0.39	0.54	1.19	0.59	14.2	18.9	2.97	1.1
	3	24.1	413.7	0.39	0.76	1.19	0.59	14.2	24	2.97	1.1
	4	24.1	413.7	0.39	0.83	1.19	0.59	14.2	27.5	2.97	1.1
	5	24.1	413.7	0.39	0.73	0.73	0.47	16	18.9	2.63	0.97
	6	24.1	413.7	0.39	0.93	0.73	0.26	16	18.9	2.63	1.0
	7	24.1	413.7	0.11	0.73	0.73	0.47	16	18.9	2.63	0.78
	8	24.1	413.7	0.39	0.83	1.19	0.59	14.2	27.5	2.97	1.1
	9	24.1	413.7	0.39	0.83	1.19	0.59	14.2	27.5	2.97	1.13
	10	24.1	413.7	0.39	0.73	0.73	0.47	16	18.9	2.63	0.95
Calvi <i>et al.</i> [C6]	C2	23.9	385	0.12	0.68	0.5	0.5	16.7	41.3	2.36	1.0
	C4	23.9	385	0.12	0.68	0.32	0.32	16.7	41.3	2.36	1.2
Hakuto <i>et al.</i> [H1] (2000)	O1	41.0	325	0	0.69	1.21	0.6	12.5	21.0	2.03	0.72
	R2	43.0	330	0	0.69	0.68	0.34	29.2	25.0	0.72	0.23
	R3	42.0	325	0	0.69	1.21	0.6	29.2	21.0	0.86	0.27

	O4	53.0	308	0	1.35	1.21	1.21	25	17.9	0.85	0.57
	O5	33.0	306	0	1.52	1.07	1.07	18.8	17.9	1.42	0.74
Wu, Y.M. <i>et al.</i> [W1] (2001)	O1	32.3	503	0		0.63	0.38	18.8	24	0.53	0.84
	M1	32.0	503	0		0.63	0.38	18.8	24	0.53	0.85
	O2	32.5	503	0		1.17	0.92	36	24	0.28	0.61
	M2	30.3	503	0		1.17	0.92	36	24	0.28	0.62
Yin, H. <i>et al.</i> [Y1] (2000)	C1A	23.7	510	0.15	0.77	2.44	0.97	15.6	22	3.36	1.26
	C1B	23.7	510	0	0.77	2.44	0.97	15.6	22	3.36	1.33
	C4A	23.7	510	0.15	0.42	2.92	1.46	12.5	22	4.19	1.28
	C4B	23.7	510	0	0.42	2.92	1.46	12.5	22	4.19	1.29
Experiments presented in this study.	AL1	31.7	473	0	0.68	1.04	0.79	10	20	0.95	1.29
	AS1	31.4	473	0	0.58	1.04	0.79	10	20	0.95	1.51
	MAS1	31.2	473	0	0.58	1.04	0.79	10	20	0.95	1.43
	AL2	34.6	473	0	1.26	1.04	0.79	20	20	0.47	1.06
	AS2	35.8	473	0	1.26	1.04	0.79	20	20	0.47	1.05

List of symbols in **Table 8-1**:

f'_c : Concrete compressive strength
 f_y : Yielding strength of main bars
 $N/A_g f'_c$: Column axial load ratio
 BI: Bond index

ρ_{top} : Beam top reinforcing ratio
 ρ_{bot} : Beam bottom reinforcing ratio
 M_r : Column to beam flexural strength ratio

h_c/d_b : Column depth over beam bar diameter
 h_b/d_c : Beam depth over column bar diameter

Table 8-2 Parameters of interior wide beam-column joints

Reference	Specimen	f'_c (MPa)	f_y (MPa)	f_p (MPa)	N/A_g f'_c	M_r	ρ_{top} (%)	ρ_{bot} (%)	h_d/d_b	h_b/d_c	b_b/b_c	R_{anchor} (%)	BI (MPa)	$v_{jh}(\sqrt{f'_c})$
Popov <i>et al.</i> [P5] (1992)		28	137.1	137.1	0.38	3	0.92	0.61	21.9	11.6	1.8	48/48.2	0.57	0.64
Hatamoto <i>et al.</i> [H4] (1989)	WB-1	23.0	224.7	230	0	5.2	1.64	1.64	18.4	13.1	0.87	100	1.27	0.63
	WB-2					2.6	1.61	1.61			1.77	50		1.28
	WB-3					1.7	1.6	1.6			2.67	33.3		1.93
	WB-4					1.3	1.6	1.6			3.57	25		2.6
	WB-2C					2.6	1.61	1.61			1.77	50		1.28
	WB-3C					1.7	1.6	1.6			2.67	33.3		1.93
Quintero-Febres and Wight [Q1] (1995)	WF-2	24.0	287.8	272.6	0.2	2.3	0.95	0.95	31.4	19.6	1.83	50	0.94	1.38
	WF-3	25.3				2.3					1.83	33.3	0.91	1.38
	WF-4	24.0				>4					3.63	16.7	0.47	1.38
	IWB1	27.5	414		0	1.4	0.53	0.45	19.2	24	2.5	48---38	2.06	1.72
	IWB2					1.5	0.71	0.61			1.86	48---38	2.06	1.72
	IWB3					1.7	0.72	0.58			2.36	52---40	1.64--- 2.47	1.6
Stehle <i>et al.</i> [S4] (2001)	1	41-47	444	350	$\sqrt{\quad}$	0.86	1.26	0.5	20.8- 10.0	16.7	4.8	20-25	1.67- 3.46	1.2
	2	38-41			$\sqrt{\quad}$	0.86							1.73- 3.6	1.2
Experiments presented in this study..	IWB-1	64.1	524	250	0	>4	1.43	1.0	40	12	2.67	38-44	0.81	1.4
	IWB-2	66.0	524	250	0	>4	1.43	1.0	40	12	2.67	38-44	0.81	1.37
	IWB-3	47.9	524	250	0	1.5	1.43	1.0	18.75	12	0.89	100	2.02	1.14

Table 8-3 Parameters of exterior wide beam-column joints

Reference	Specimen	f_c' (MPa)	f_y (MPa)	f_p (MPa)	N/A_g f_c'	M_r	ρ_{top} (%)	ρ_{bot} (%)	h_c/d_b	h_b/d_c	b_b/b_c	R_{anchor} (%)	BI (MPa)	$v_{jh}(\sqrt{f_c'})$
Hatamoto <i>et al</i> [H4] (1991)	WF-1	24.0	287.8-235.	272.6	0.2	3.45	0.95	0.95	31.4	19.6	1.83	50	0.94	0.7
	1	26.8-28.4	308-432	235	0.025	1.46	0.78	0.61	22.4	16	2.43	33-43	1.29	1.13
2	26.8-28.4	1.64				0.79	0.59	22.4	2.14		50-33	1.29	1.0	
3	19-37.8	1.5				0.76	0.60	18.7-22.4	2.43		56-44	1.12-1.34	0.94	
4	37.8	1.17				0.98	0.81	22.4-28	2.43		35-35	0.89-1.12	1.43	
LaFave and Wight. [L5] (1999)	EWB-1	24.8-26.9	308-442	250	0	1.6	0.42	0.52	26.7-32	16	2.43	44-47	0.97-1.16	1.2
	EWB-2	30.3-38.6	308-442			1.45	0.93	0.76	26.7-32	16	2.43	44-51	0.87-1.05	1.3
	EWB-3	34.5-35.1	323-342			1.45	0.93	0.76	26.7-32	16	3.08	20-33	0.86-1.03	1.5
Experiments presented in this study..	EWB-1	64.0	524	250	0	>4	1.43	1.0	40	12	2.67	38-44	0.82	0.7
	EWB-2	64.9	524	250	0	>4	1.43	1.0	40	12	2.67	38-44	0.81	0.64
	EWB-3	45.2	524	250	0	1.5	1.43	1.0	18.75	12	0.89	100	2.08	0.6

List of symbols in **Tables 8-2 and 8-3**: f'_c : Concrete compressive strength f_y : Yielding strength of main bars f_p : Yielding strength of joint shear reinforcement $N/A_g f'_c$: Column axial load ratio

BI: Bond index

 ρ_{top} : Beam top reinforcing ratio ρ_{bot} : Beam bottom reinforcing ratio M_r : Column to beam flexural strength ratio R_{anchor} : Percentage of beam longitudinal bars anchored in the column h_c/d_b : Column depth over beam bar diameter h_b/d_c : Beam depth over column bar diameter b_b/b_c : Beam width over column width

8.3.3 Finite Element Modelling

8.3.3.1 Analytical Models and Material Properties

The preceding review of experimental data shows conclusively that the number of parameters affecting the behaviour of beam-column joints exceeds the breadth of the available database. To compensate for the limitations of the experimental studies, analytical modelling is used to supplement the parametric study. In addition, a finite element analysis is adopted in this study, which was carried out by a finite element analysis program DIANA [D3].

The finite element models studied in this section were three-dimensional idealization of beam-column connections with geometries similar to that of beam-column assemblies tested in previous experiments. The prototype of conventional beam-column joint's models were derived from the Specimen C2 tested by Calvi *et al.* [P15] while the wide beam-column joint's models were set up based on the tested specimens in this study with some modifications.

In the conventional beam-column joint's models, the section of columns and beams were 200 mm×200 mm and 330 mm (depth) ×200 mm (width), respectively. The columns were reinforced with six $\phi 8$ rebars leading to a column longitudinal reinforcing ratio being 0.8%. In all the models, $\phi 4$ closed hoops with a spacing of 115 mm were used as beam transverse reinforcement, while the columns were reinforced with the same transverse reinforcement at spacing of 135 mm.

In the wide beam-column joint's models, the section of main beams was 800mm×300mm. The column width ranged from 400 mm to 600 mm. Both a square column section and a rectangular column section were used in this analysis. Other details will be introduced in the following sections.

Finite element modelling and constitutive laws have been introduced in the above chapters with the material properties used in the analysis being shown in **Table 8-4**.

Table 8-4 Material properties

Properties	Conventional Beam-Column Model (MPa)	Wide Beam-Column Model (MPa)			
E_s	210000	192500			
f'_c	24	23	30	40	70
f_t	1.6	1.57	1.75	1.97	2.46
f_y	385	345			
f_u	451	517			

8.3.3.2 Parameters Investigated for Conventional Beam-Column Joints

In this study, the concerns regarding the influence of parameters on the behaviour of conventional beam-column connections focus on the effect of column axial loads, beam longitudinal reinforcing ratios and presence of slabs.

Column Axial Load

It is clear in **Table 8-1** that in some of the previous tests, the column axial load was applied and was kept constantly, while in other tests the column axial load was not applied. Usually, the column axial load is expressed as a percentage of the product of the column gross area and concrete compressive strength. In the current study, the column axial loads were applied with a magnitude ranging from 0 to $0.4A_g f'_c$.

Beam Longitudinal Reinforcing Ratio

In the experimental database, the beam longitudinal reinforcing ratio ranged from 4.38% to 0.64% while in the analytical study, a ratio of beam bottom bar area to top bar area A_s/A'_s , rather than the total beam reinforcing ratio is of concern. Three A_s/A'_s ratios

are considered in the current study, which are 0.75:1, 1:1 and 1:2 resulting in the total beam reinforcing ratios being 0.73%, 1.0% and 0.73%, respectively.

Presence of Slab

In the experimental database, most of the specimens were tested without consideration of slab effects except Specimens AS1, MAS1 and AS2, which were tested and presented by the author in this thesis. In the analytical study, the slab width equals $b_b + 2kh_b$, in which b_b is the beam width and h_b is the beam depth and k ranges from zero to four.

8.3.3.3 Parameters Investigated for Wide Beam-Column Joints

The parameters that have a significant influence on the behaviour of wide beam-column connections include column axial loads, beam to column width ratios, beam longitudinal bar anchorage ratios and the effect of transverse beams. The range of the parameters considered in this study is shown in **Table 8-5**.

Table 8-5 Parameters investigated in the analytical study for the wide beam-column joint's models

	Description	Ranges investigated
N_c	Column axial load	0 to $0.6 A_g f'_c$
b_w / b_c	Beam width to column width ratio	2.0 to 4.0
$R_{anchor.}$	Beam longitudinal bar anchorage ratio	33% to 75%
f'_c	Concrete compressive strength	C20 to C70

Besides the examination of these parameters, other factors such as the effect of joint confinement, bond condition and the effect of transverse beam are also concerned. In addition, the interrelated effect of these parameters on the wide beam-column joint's behaviour will also be considered.

In order to study the effect of transverse beam on the performance of wide beam-column connections, the additional models containing four exterior wide beam-column

connections were set up with a different type of transverse beams. **Figure 8-1** shows the analytical models representing four types of transverse beam configuration.

Figure 8-1 Analytical models for the study of transverse beam effect.

In Model-A, there was no transverse beam casting into the connection. The outside beam portions were confined by beam transverse reinforcement. In Model-B, a transverse beam with a cross section of $300 \times 400 \text{ mm}^2$ was built into the connection (coloured in blue). The transverse beam in Model-C had a conventional cross section at $400 \times 200 \text{ mm}^2$ with the reinforcing details being similar to those of the transverse beam in Model-B. On one hand, due to a relatively small section area, the reinforcing ratio of the transverse beam in Model-C was slightly higher than that of transverse beam in Model-B. On the other hand, the torsional stiffness of the transverse beam in Model-C was higher than that of transverse beam in Model-B. The transverse beam in Model-D had an identical geometry

with that of the main beam; therefore, it was also a wide beam with a beam to column width ratio being of 2.0. The reinforcing ratio of the transverse beam was similar to the main beam while some of the reinforcing bars were placed outside the column core. Among these transverse beams, the transverse beam in Model-D had the largest cross section area.

Beam Longitudinal Bar Anchorage Ratio

The beam longitudinal bar anchorage ratio has a significant influence on the behaviour of wide beam-column joints. In the current study, four cases with an anchorage ratio ranging from 33% to 75% were investigated. The modelling criteria adopted in this study are summarized as below:

- (1) The beam across section, the beam longitudinal reinforcing ratio and the column to beam flexural strength ratio should remain equal among these four models.
- (2) A smaller bar diameter should be used for the beam to satisfy the desired anchorage ratio.
- (3) When the column width increases, the column depth and column longitudinal reinforcement should vary to obtain a uniform column to beam flexural strength ratio.

Figure 8-2 shows the different beam longitudinal bar configurations for the studying of the anchorage ratio effect on the behaviour of wide beam-column joints.

Figure 8-2 Different beam longitudinal bar anchorage ratios used in the parametrical study

8.3.3.4 Loading Sequences

Although under laboratory conditions some specimens were loaded by displacing the column ends, in the analysis the beam ends were displaced to minimize an $P - \Delta$ effect. Column axial loads were applied in a full magnitude from the first load increment, while beam end loads were applied incrementally by means of a vertical displacement. Thus, the inter-storey drift ratio was defined as the ratio of the beam tip displacement to the beam length and the load versus deformation relationship was presented in terms of the vertical load on beam end versus the drift ratio defined as above.

8.4 Discussion of Database Results

8.4.1 Results of Conventional Beam-Column Joints

8.4.1.1 Effect of Column Axial Load

The column axial load is believed to be beneficial to the joint shear resistance because it either confines the joint core, or equilibrates part of the diagonal concrete compression strut that formed inside the joint because of joint shear actions. To study the effect of column axial load to the magnitude of the joint shear resistance, the experimental joint shear stresses versus column axial loads normalized by $A_g f'_c$ is plotted in **Figure 8-3**.

Figure 8-3 shows that the experimental data are considerably scattered. The column axial load in the range of $0.1A_g f'_c$ to $0.4A_g f'_c$ does not influence the magnitude of joint shear stresses. On the contrary, a larger magnitude of joint shear stresses can be obtained even when the column axial load is not applied. Similar phenomena was reported by Pantazopoulou [P14], Bakir [B5] and Yin [Y1]. It was summarized by these researchers that deformability rather than strength would be affected by the column axial load; however, currently available experimental information regarding deformation is very limited and does not warrant generalized conclusions. [P14]

Since the tested specimens were different in geometry and reinforcing details, a firm relationship between the applied column axial loads and nominal joint shear stresses was not directly comparable. In the parametric study, this can be done by applying different magnitude of column axial loads while the other variables are fixed. **Figures 8-4 to 8-6** illustrate the nominal joint shear stresses versus different column axial loads through finite element analyses.

In **Figure 8-5**, the analytical models were pure beam-column joints with the beam bottom to top bar area ratios being 0.75:1, 1:1 and 1:2. When the column axial loads were not applied, the maximum nominal joint shear stresses of these analytical models ranged from $0.4\sqrt{f'_c}$ to $0.5\sqrt{f'_c}$ (f'_c unit in MPa). When the magnitude of the column axial load increased to $0.1A_g f'_c$ MPa, the maximum nominal joint shear stresses increased markedly to $0.87\sqrt{f'_c}$, $0.97\sqrt{f'_c}$ and $0.73\sqrt{f'_c}$, respectively. The joint shear demand of the analytical model with a A_s / A'_s ratio of 1:1 increased with the applied column axial loads; whereas the other two models reached their maximum joint shear stresses at the column axial load of $0.1 A_g f'_c$, which the joint shear demand of these two models decreased gradually.

In **Figure 8-6**, the analytical models were beam-column joints with a slab where the slab width equalled the beam width plus three times beam depth at each side of the beam faces and where the A_s / A'_s ratios were 0.75:1, 1:1 and 1:2, respectively. When the column axial loads were not applied, the maximum nominal joint shear stresses of these analytical models ranged from $0.3\sqrt{f'_c}$ to $0.52\sqrt{f'_c}$ (f'_c unit in MPa). The joint shear demand in the first model, which had a A_s / A'_s ratio of 0.75:1 increased constantly until a column axial load of $0.2 A_g f'_c$ was applied at the point where the model gained its maximum joint shear stress at $0.65\sqrt{f'_c}$. When the magnitude of the column axial load was $0.1A_g f'_c$, the second

model, which had a A_s / A'_s ratio of 1:1 reached its maximum joint shear stress at $0.55\sqrt{f'_c}$. Meanwhile, the third model with a A_s / A'_s ratio of 1:2 also obtained its maximum joint shear stress at this stage with the value being $0.58\sqrt{f'_c}$. All the analytical models lost their joint shear strength at a higher column axial load level.

It can be seen from the analytical results that, for the beam-column joints without joint transverse reinforcement, the column axial load with a magnitude from $0.1A_g f'_c$ to $0.2A_g f'_c$ would significantly influence joint shear stresses; however, the joint shear strength would decrease as the column axial load increases from $0.2A_g f'_c$ to $0.4A_g f'_c$.

By employing a regression analysis, the nominal joint shear stresses versus column axial loads relationship was obtained (see **Figure 8-7**), which can be expressed as:

$$v_{jh} = \left(0.4 \frac{N}{A_g f'_c} + 0.52 \right) \sqrt{f'_c} \quad (8-9)$$

This equation reflects the influence of the magnitude of column axial loads along with the quantity of slab participation on the joint shear behaviour. When the column axial loads were not applied, the minimum joint shear stress of $0.52\sqrt{f'_c}$ can be expected. Moreover the maximum joint shear stress of $0.92\sqrt{f'_c}$ can be taken as the upper limit for a beam-column joint without joint transverse reinforcement and with consideration of presence of slab.

Figure 8-3 Nominal joint shear stress versus column axial load relationship via experimental approach

Figure 8-4 Nominal joint shear stress versus column axial load relationship via analytical approach

Figure 8-5 Nominal joint stress versus column axial load relationship for the analytical models without slab.

Figure 8-6 Nominal joint stress versus column axial load relationship for the analytical models with slab

Figure 8-7 Nominal joint shear stress versus column axial load relationship via regression analysis.

8.4.1.2 Effect of Slab Participation

Since most of the tested specimens were constructed without a slab, the slab influence on seismic behaviour of the beam-column joints was investigated by the analytical results.

Figures 8-8 to 8-16 show the slab influence on the overall behaviour of the analytical models subjected to difference column axial loads.

It can be seen from these figures that the participation of slabs enhanced the stiffness of the analytical models indeed and increased the overall strength of the models before a larger deformation level of 2% was reached. When the slab width was larger than $b_b + 2 \times 2b_h$, the difference between each curve was not much evident, which implied that the effective slab width should be the beam width plus twice beam depth on each side of the beam faces.

Figure 8-8 Slab influence on the overall behaviour of analytical models with a A_s / A'_s ratio of 0.75:1, which subjected to a column axial force at $0.2A_g f'_c$.

Figure 8-9 Slab influence on the overall behaviour of analytical models with a A_s / A'_s ratio of 1:1, which subjected to a column axial force at $0.2A_g f'_c$.

Figure 8-10 Slab influence on the overall behaviour of analytical models with a A_s / A'_s ratio of 1:2, which subjected to a column axial force at $0.2A_g f'_c$.

Figure 8-11 Slab influence on the overall behaviour of analytical models with a A_s / A'_s ratio of 0.75:1, which subjected to a column axial force at $0.3A_g f'_c$.

Figure 8-12 Slab influence on the overall behaviour of analytical models with a A_s / A'_s ratio of 1:1, which subjected to a column axial force at $0.3 A_g f'_c$.

Figure 8-13 Slab influence on the overall behaviour of analytical models with a A_s / A'_s ratio of 1:2, which subjected to a column axial force at $0.3 A_g f'_c$.

Figure 8-14 Slab influence on the overall behaviour of analytical models with a A_s/A'_s ratio of 0.75:1, which subjected to a column axial force at $0.4A_g f'_c$.

Figure 8-15 Slab influence on the overall behaviour of analytical models with a A_s/A'_s ratio of 1:1, which subjected to a column axial force at $0.4A_g f'_c$.

Figure 8-16 Slab influence on the overall behaviour of analytical models with a A_s / A'_s ratio of 1:2, which subjected to a column axial force at $0.4A_g f'_c$.

Figure 8-17 Nominal joint shear stress versus slab width relationship via regression analysis

Figures 8-17 shows the relationship between the nominal joint shear stresses and slab widths. It is clear that the presence of a slab will affect joint shear strength to some extent, but the magnitude is not significant

8.4.1.3 Effect of Beam Reinforcing Ratio and Reinforcing Details

Figures 8-18 and **8-19** show the influence of beam reinforcing details on the joint shear stresses. In **Figure 8-18** the relationship is presented in terms of the nominal joint shear stresses versus beam bottom reinforcement to top reinforcement ratios; however, there is no firm relationship can be set up from this figure. When the A_s / A_s' ratio equals 1:1, no joint shear stresses were found higher than $1.2\sqrt{f_c'}$ MPa from the analytical results, which implied that a symmetrically reinforced beam member could be critical in joint shear resisting.

Figure 8-18 Nominal joint shear stress versus beam bottom reinforcement to top reinforcement ratio relationship via both experimental and analytical approaches.

In **Figure 8-19**, the relationship is presented in terms of the nominal joint shear stresses versus total beam longitudinal reinforcing ratios. It is evident that the joint shear stresses will be increased with the increasing of the total beam longitudinal reinforcing ratios. By employing a regression analysis this relationship can be expressed as:

$$v_{jh} = \left[0.22 \times 100 \left(\frac{A_s + A'_s}{b_b h} \right) + 0.45 \right] \sqrt{f'_c} \quad (8-10)$$

$$\text{or } v_{jh} = \left[0.22 \times 100 \rho (1 + \lambda) + 0.45 \right] \sqrt{f'_c} \quad (8-11)$$

where, ρ is the ratio of beam bottom reinforcement to beam section " $\frac{A_s}{b_b h}$ "; λ is the ratio

of beam top to bottom reinforcement area " $\frac{A'_s}{A_s}$ ".

Figure 8-19 Nominal joint shear stress versus beam longitudinal reinforcing ratio relationship via regression analysis.

8.4.1.4 Effect of Bond Condition

The contribution to this topic has been made by researchers from Kajima Institute [H4]. It was found from their experimental investigations that specimens exhibited with pinched hysteretic loops in spite of the higher column-to-beam flexural strength ratios. These results clearly indicated the importance of anchorage parameter on the overall performance of the specimens. To evaluate the severity of bond stresses, Kitayama *et al* [K3] proposed a bond index of beam longitudinal reinforcement with an assumption that the beam longitudinal reinforcement at the tension and compression faces of the joint yield simultaneously. The bond index is described as:

$$BI = \frac{u_b}{\sqrt{f'_c}} = \frac{f_y (d_b / h_c)}{2\sqrt{f'_c}} \quad (8-12)$$

where:

u_b : The maximum bond stress of the beam longitudinal bars over the column width

d_b : Beam bar diameter

h_c : Column depth

f_y : The yield strength of the beam longitudinal bars (unit in MPa)

f'_c : Concrete compressive strength (unit in MPa)

Based on the definition, the AIJ [A7] provision limits it as 1.4.

Figure 8-20 Nominal joint shear stress versus bond index relationship for the previously tested conventional beam-column joints

Application of the bond index to evaluate the failure mechanism of beam-column joints was summarized by Yin [Y1] in her database study. It was recommended that: an upper boundary of the bond index equal to 1.0 for lightly reinforced joints and 1.5 for ductile joints is helpful in preventing possible joint shear failure. When BI is greater than 2.0, joint shear failure cannot be avoided in all cases. This was validated in current study. When most of the tested specimens had a bond index larger than 2.0, the joint shear failure was found during the test. Some specimens even with a bond index less than 1.0 could not prevent joint shear failure. **Figure 8-20** shows the relationship of nominal joint shear stresses versus bond indexes.

8.4.2 Results of Wide Beam-Column Joints

8.4.2.1 Effect of Beam Longitudinal Bar Anchorage Ratio

In the Hatamoto [H4] test, a significant pinching behaviour of WF-3, which had less magnitude in its anchorage ratio, was found and as a result, Hatamoto *et al* [H4] indicated

that this phenomenon might attribute to the higher shear force and moment transferred to the column by torsional stresses in the outside beam portions. Therefore, he suggested that the amount of beam longitudinal bars placed in the outside beam portions should be limited. As early as 1992, Paulay and Priestley [P7] also recommended that at least three-fourths of the beam longitudinal bars should pass through the column core.

Changing the anchorage ratio of the beam longitudinal bars passing through the column core will introduce positive as well as negative effects on the joint shear behaviour. On one hand, a greater number of beam longitudinal bars passing through the joint core are helpful in resisting joint shear input because the participation of these bars helps to relieve the burden of the joint panel concrete and joint shear reinforcement. On the other hand, excessive longitudinal bars passing through the joint core induce higher shear stresses in the joint panel. However, increasing the beam longitudinal bar anchorage by percentage will reduce the shear forces and moment transferred by torsional moment. Hence, the joint behaviour of a wide beam-column joint is dominated by several parameters as well as their interaction, which cannot be evaluated via a single parameter alone.

Figure 8-21 Nominal joint shear stress versus beam longitudinal bar anchorage ratio relationship via experimental approach

Figure 8-21 shows the results of the previous experimental investigations. As the anchorage ratio increased, the induced joint shear forces become higher and all the joint shear stresses lie on or above the ACI-ASCE 352 [A3] recommended limits. No relationship can be set up between the joint shear stresses and beam longitudinal bar anchorage ratio. When this ratio is lower, a large quantity of joint shear forces may attribute to the beam longitudinal bars anchored in the transverse beam rather than those anchored in the joint core. It can be expected that this is only true when the transverse beam maintains its torsion capacity to transfer the shear forces and induced moment. Once the transverse beam loses its capacity, the majority of the joint shear will mainly attribute to the induced shear forces by the beam longitudinal bars passing through the joint core. At this moment, heavy reinforcement will cause the joint to be relatively weaker than the framing members will. Consequently, the beam hinging failure mechanism may be changed to an unfavourable joint shear failure mechanism; however, this is not an issue in the current analysis.

Figure 8-22 Nominal joint shear stress versus beam longitudinal bar anchorage ratio relationship via analytical approach

The analytical results in this study show that the overall behaviours in terms of horizontal displacement versus storey shear force are identical. In other words, changing the anchorage ratio of the beam longitudinal bars passing through the joint core does not affect the hysteretic behaviour of wide beam-column connections. This is evident because it does not increase the flexural capacity of beams and the column to beam flexural strength ratios. **Figure 8-22** illustrates the influence of beam longitudinal bar anchorage ratios on the joint shear behaviour. It can be seen that the joint shear stresses increase with the increasing of the beam longitudinal bar anchorage ratio, but how much reinforcement should be placed inside the column core has not yet been made very clearly by these limited experimental and analytical investigations and this area needs further study.

8.4.2.2 Effect of Bond Condition

Figure 8-23 shows the relationship of nominal joint shear stresses versus bond indexes based on the previously tested wide beam-column joints. There was no relationship setup between the BI and failure mechanisms of these specimens in the literature. However, it can be seen that most of the tested exterior wide beam-column joints have a BI value between 1.0 to 1.5 and the joint shear stresses lie above the ACI-ASCE 352 recommendation indicating which says the joint shear stress of $1.0\sqrt{f'_c}$ is possibly conservative for design of an exterior wide beam-column joint. For the interior wide beam-column joints, if the bond index value is lower, the joint shear stresses will lie on or below the ACI-ASCE 352 recommended limit; but when the bond index value is higher; some of the interior wide beam-column joints' shear stresses will be much higher.

Further discussion regarding of the bond behaviour has been made by Quintero-Febres *et al* [Q1]. It was found through their experimental study that a large column to beam flexural strength ratio would not guarantee a robust hysteretic behaviour to prevent slipping because of the poor bond condition. In addition, the bond conditions in the outside beam portions were different. The anchorage parameter for the beam longitudinal

bars passing through the outside beam portions should be expressed in terms of the transverse beam width over the beam bar diameter rather than the column depth. However, the anchorage of the beam longitudinal bars passing through the transverse beam is only reliable before the transverse beam reaches its cracking strength. The results obtained from this analysis indicated that the distribution of the bond force does not seem to be an important variable associated with the strength of interior or exterior wide beam-column joints.

Figure 8-23 Nominal joint shear stress versus bond index relationship for the previously tested wide beam-column joints

8.4.2.3 Effect of Column Axial Load

One of the most controversial issues in the design of the wide-beam column joints is the column axial load effect on the strength of the joints. Because of a compression load on the column, the neutral axis depth in the column will increase which leads to an enhancement to the diagonal concrete compression strut mechanism in the joint panel. In addition, the column axial load helps to prevent the diagonal concrete cracking and hence improves the beam longitudinal bar anchorage condition.

The effect of column axial loads on the performance of beam-column joints was firstly studied by Paulay *et al* [P7, P10] and since then further extensive investigations have been conducted on this topic. However, at what column axial load level do the joint shear strength and bond condition become a significant influence is an argued issue amongst different researchers. For example, in the NZS 3101 [C1] design standard, the contribution of the main strut mechanism is ignored unless a significant column axial load greater than $0.1A_g f'_c$ exists. Meinheit and Jirsa [M3] observed that the ultimate shear strength of a joint was unaffected by the column compression load which varied from 0 to $0.05A_g f'_c$ MPa. Kitayama *et al* [K3] indicated that column axial stresses smaller than $0.3f'_c$ did not exhibit a beneficial effect on the bond resistance along the beam bars within a joint, and that anything smaller than $0.5f'_c$ did not influence the joint shear strength. Shiohara [S3] also pointed out that the column axial load did not affect the shear strength of ductile joints. On the other hand, a column axial load with a range from 0 to $0.15A_g f'_c$ did not influence the shear strength of lightly reinforced beam-column joints.

It should be noted that these conclusions were based on the experimental and analytical investigations of typical beam-column joints and that due to the limited test data, the column axial load effect on the wide beam-column connections has so far not been clearly addressed. In the current study, the effect of the column axial load was studied by using the finite element approach, in which the column axial load ranging from 0 to $0.6A_g f'_c$ was applied to the interior and exterior wide beam-column joints' models. **Figures 8-24** shows the effect of the column axial loads on the joint shear stresses of the analytical models. It is clear that when the column axial load equals $0.2 \frac{N_c}{A_g f'_c}$, most of the analytical models have a maximum joint shear stress lying on the ACI-ASCE 352 limits; however, the joint shear stresses decrease with the increase of the column axial loads.

Figure 8-24 Nominal joint shear stress versus column axial load relationship for the analytical models with column axial load level ranging from 0 to $0.6 \frac{N_c}{A_g f_c}$

Figure 8-25 Nominal joint shear stress versus column to beam flexural strength ratio relationship for the previously tested wide beam-column joints

Results from the analyses showed that the models experiencing column axial loads had their peak storey shear strength occurring at a similar storey drift with the values lowering as the column axial load increased. After the peak strength, the models with a column axial load demonstrated more rapid strength degradation though in general an axial load of $0.2A_g f'_c$ had a limit influence on the overall behaviour. Strength and stiffness degradation due to the axial load and the subsequent $P-\Delta$ effect could be negligible, but the joint shear capacity was sensitive to the column axial load when its magnitude was high. When comparing the overall performance of the interior and exterior wide beam-column connection's models, it was found that the column axial load effect on the exterior beam-column joints was more significant than that on the interior beam-column joints. On one hand, the exterior beam-column connection's models lost 20% of their load-carrying capacities as early as a 2% drift ratio was reached, which implied the failure of these models. On the other hand, the strength degradation in the interior wide beam-column connections did not result in a failure of the models and they retained their load-carrying capacities being higher than the theoretical values until a storey drift ratio of 4% was reached.

8.4.2.4 Effect of Transverse Beam

As mentioned above, the analytical models with different transverse beam configurations were concerned in this study with the main variable being the dimension of the transverse beam.

It was found that all the joint shear stresses were inferior to the ACI 318 [A2] provisions. Among these four models, joint shear stresses in Models A and B were identical. In Model-A, there was no transverse beam framing into the connection and the outside beam portions were confined by the beam transverse reinforcement. Therefore, the shear forces and bending moment generated by the beam longitudinal bars, which were placed in the outside beam portions, could only be transferred in the joint core to some extent.

Although Model-B contained a transverse beam, the relative smaller transverse beam section could not significantly contribute to the joint shear performance. The section of the transverse beam of Model-C was smaller than that of Model-B, but the torsion inertia was slightly higher, which could induce a larger magnitude of shear forces into the joint. Model-D had the largest transverse beam section and therefore the load-carrying capacity of this model was the highest among these four models. It can be concluded from the current study that a well-reinforced transverse beam with a large cross section may contribute to the superior behaviour of the beam longitudinal bars outside the column. This can be achieved by increasing the reinforcing ratio of the transverse beam or the dimension of the transverse beam, which leads to an increase in the torsion strength and stiffness of the transverse beam. However, the significant role of a transverse beam can only be expected before it reaches the cracking torsional strength.

Similar phenomenon were reported in the previous investigations by Hatamoto *et al.* [H4] that the transverse beam reinforcing did not increase the total storey shear force but it did alleviate some of the low displacement pinching. In addition, joint shear was also improved though the magnitude was negligible. It was summarized that the transverse reinforcement plays a more important role in the behaviour of the connections with the increase of the beam to column width ratio.

The tests conducted by Gentry [G2] demonstrated that the behaviour of exterior beam-column connections was characterized by the ability of the transverse beam to transfer the wide beam's flexural forces to the column. If the transverse beam cracked significantly in torsion, the specimen lost its stiffness and the wide beam was not able to fully develop its plastic hinge. It was also concluded that the behaviour of exterior wide beam-column connections was controlled by the torsion strength of the transverse beam, whereas interior wide beam-column connections did not display this same failure.

No specimens were reported to have joint shear failure when the beams are framed into all vertical faces of the joint. It is evident that the transverse beams confine the joint by their longitudinal bars, which are anchored inside the joint in the transverse directions, and by effectively increasing the volume of the joint concrete to actively participate in the joint shear resistance. In a wide beam-column connection, the outside beam portions act as a transverse beam.

8.4.2.5 Discussion of the Influence of Parameters on Joint Shear Behaviour of Wide Beam-Column Joints

The influence of parameters on the joint shear behaviour has been discussed in the above sections and the relationship of nominal joint shear stresses versus various design parameters has been plotted. Due to the limited data available on wide beam-column joints however, the extent of the influence of the parameters still requires further investigations. In general, the magnitude of the joint shear stresses is highly dependent on the definition of an effective joint volume and the nominal joint shear stresses should be calculated on the area defined by ACI-ASCE 352 [A3] rather than a full joint area including the wide beam region. According to this recommendation, when a beam width is wider than that of the column, the effective joint width should be taken as the beam width or the column width plus 0.5 times the column depth whichever is smaller. However, even with consideration of column cross section alone, the designed and measured joint shear stresses of the exterior wide beam-column joints is quite a bit lower than the ACI 318 [A2] provision. On the other hand, the magnitude of joint shear stresses of an interior wide beam-column joint lies on or well beyond the ACI 318 [A2] provision upper bound. A constant trend is found based on this preliminary parametric study that the interior wide beam-column joints could indeed withstand higher shear input.

The author suggested that when designing of a wide beam-column joint, $1.25\sqrt{f'_c}$ might be a reasonable limit in preventing the maximum joint shear stress input for interior wide

beam-column joints. Moreover, $1.0\sqrt{f'_c}$ might be a conservative indicator for the maximum joint shear stress that can be achieved in exterior wide-beam-column joints.

8.5 Summary

The study described in this chapter was devoted to improve the understanding of joint shear resistance. The influence of several key variables on the behaviour of beam-column joints was evaluated by the results of a database study and a finite element analysis. Based on which, the following conclusions and design recommendations are made:

- (1) The parameters investigated in this chapter significantly influence the seismic behaviour of lightly reinforced beam-column joints as well as wide beam-column joints with the effect of these parameters being interrelated.
- (2) Beam-column joints in a gravity-load-designed (GLD) or a lightly reinforced concrete (LRC) frame are sensitive to the column axial loads. Column axial loads with a magnitude ranging from $0.1A_g f'_c$ to $0.2A_g f'_c$ will evidently enhance the joint shear resistance; however, the influence of a larger magnitude of column axial load is rather limited. In addition, the column axial load has significant effect on the behaviour of the wide beam-column joint, where $0.2A_g f'_c$ is a reasonable limit in obtaining the desired performance.
- (3) In a conventional reinforced concrete frame, slab participation interrelated with the other parameters will enhance the overall performance of the beam-column joints. When assessing the performance of a GLD or LRC frame building, the slab effect cannot be neglected but the effective slab width should be carefully determined. The parametric study in this chapter and the experimental study carried out in Chapter 5 show that, for a GLD or LRC frame, the effective slab width cannot exceed the beam width plus two times beam depth on the each side of the beam.

- (4) In general, interior wide beam-column joints represent desired behaviour with the joint shear stresses lying on the design code provisions. Exterior wide beam-column joints performed with higher joint shear stresses than it can be expected. Design limits on the exterior wide beam-column connections can be relaxed to some extent.
- (5) The $1.25\sqrt{f'_c}$ might be a reasonable limit to prevent the maximum joint shear stress input for interior wide beam-column joints. On the other hand, $1.0\sqrt{f'_c}$ is a conservative indicator of the maximum joint shear stress that can be achieved in exterior wide-beam-column joints and instead, $1.25\sqrt{f'_c}$ is suggested in this study.
- (6) Bond condition is an important factor when the wide beam-column connections are considered because the wide beam-column joint's behaviour is more sensitive to the bond conditions though further studies are needed on this issue.

Chapter 9

Summary and Recommendations

9.1 Introduction

The study presented in this thesis is to investigate the seismic behaviour of reinforced concrete moment-resisting frame structures and beam-column joints, which were designed and constructed without any seismic provisions. The attention is placed on the behaviour of GLD frames and the frame with non-standard reinforcing details, which are typically used in Singapore and are considered to be inadequate to resist earthquake actions. Therefore, this study attempts to assess the performance of such kind of buildings under a low to moderate earthquake attack.

9.2 Seismic Assessments of GLD Reinforced Concrete Frames.

The assessments of seismic performances of GLD reinforced concrete frames through experimental approaches by other researchers were reviewed and an analytical investigation was conducted in this research via finite element analysis.

Both the experimental and analytical results showed that the GLD RC frames could sustain their load-carrying capacities before a drift ratio of 2.0% being reached; however, a large magnitude of joint shear demand was found at an early stage resulting in an identification of joint shear failure particularly in exterior joints. In a GLD RC frame, a joint hinge mechanism or a combined mechanism rather than a pure soft-storey mechanism is expected to develop during an earthquake action. Extensive joint damage occurring at an early stage

can lead to different global damages and hinge patterns. The local behaviour rather than global behaviour will be improved through the modification of reinforcing details. However, this technique will lead to both favourable and unfavourable effects on the overall response because it changes the deformation demand on the beam-column joints and adjacent members.

9.3 Summary of the Investigations on Lightly Reinforced Beam-Column Joints

Tables 9-1 and **9-2** tabulate the experimental and analytical results of the conventional beam-column joints.

In the test performed by author, the specimens could maintain their strength before a storey drift ratio of 2.0% being reached and after that, the strength and stiffness degradation could not be prevented as the test proceeded.

The column to beam flexural strength ratio significantly affected the global and local behaviour of the test specimens. When the specimen was designed with a “strong beam-weak column” configuration, the deformation demand on the column was significant and resulted in a large magnitude of the contribution to the horizontal displacement and yielding in the column longitudinal bars. Once the column to beam flexural strength ratio increased such as in Specimens AL2 and AS2, the contributions due to the column and beam deformation were quite close.

The maximum nominal horizontal joint shear stress of the specimens in the first group exceeded the specified limits either by ACI 318 [A2] or by NZS 3101 [C1]. Therefore, the joint shear failure mechanism would be expected in these specimens. After diagonal tension cracking developed, tension strains were prevailed in both the beam and column bars in the joint core, which resulted in significant joint expansion. Furthermore, the

participation of slab bars induced the additional shear forces in the joint, which resulted in a requirement of joint transverse reinforcement. However, the joint shear stress demands decreased as the column to beam flexural strength ratio increased. **Table 9-2** shows the results of the nominal joint shear stresses from the experiments and analyses.

The presence of a slab affected the behaviour of the specimens by increasing the load-carrying capacity of the main beam and imposing torsional moment in the transverse beam and consequently affected the confinement of the joint.

Previous studies have indicated that, in interior beam-column joints, the effective slab width would vary from one to five times the beam depth on each side of the beam. It was found in this research that the development of an effective slab width is related to the flexural strength ratio and the global displacement demand. In the first group of the specimens, the effective slab width did not develop because the overall performance was governed by the column behaviour. In the second group, due to the “strong column-weak beam” configuration, the specimens were expected to be governed by the beam behaviour and as such the effective slab width being of one times the beam depth on each side of the beam face plus the beam width was found in Specimen AS2. However, such an effective slab width was obtained at a higher deformation level.

In the first group of the specimens, the column depth to beam bar diameter ratio did not satisfy the design limits by ACI 318 [A4]. As a result, tension strains were found in either the joint core or the compression zone of the beam and yielding penetration in the joint core could not be prevented. Comparing the measured strains in the beam longitudinal bars of Specimens AL1 and AL2, it was found that increasing the column depth to beam bar diameter ratio will improve the strain distributions in the beam bars. However, in Specimen AS2, the beam top bars in the compression zone were still found in tension during the test indicating that the ACI 318 [A4] limits may not be strict enough to prevent the slippage.

Table 9-1 Summary of the experimental and analytical results of the conventional beam-column joints

Test Specimen	Column to beam flexural strength ratio	Column to beam bar diameter ratio	Predicted yield displacement (mm)	Displacement at first yield (mm)	Theoretical capacity	Predicted capacity with full slab participation	Maximum capacity in the test (kN)	Maximum capacity from FEM analysis (kN)	Member's contribution to overall deformation demand at 2% drift	
									Beam flexure	Column flexure
AL1	0.68	10/1	12.5	37	±60.6	—	±53	+65 / -57	5.5%	14.5%
AS1	0.68	10/1	12.5	19.6	±63.1	±97.3	+80 / -70	+82 / -70	5%	17.5%
MAS1	0.68	10/1	12.5	23.8	±63.1	±97.3	+73.8 / -69.6	+81.2 / -70	6%	30%
AL2	1.26	20/1	6.75	21	±63.9	—	+82 / -75	+73.8 / -68	10%	6.5%
AS2	1.26	20/1	6.54	14	±66.7	±102.7	+98 / -83	+106 / -96	18.9%	10%

Table 9-2 Results of nominal joint shear stress from experiments and analyses

Model	Joint shear stress (Experimental)			Joint shear stress (Analytical)			Joint shear stress (Analytical) Assume full slab participation		
	(MPa)	f'_c (MPa)	$\sqrt{f'_c}$ (MPa)	At cracking $\sqrt{f'_c}$ (MPa)	(MPa)	f'_c (MPa)	$\sqrt{f'_c}$ (MPa)	(MPa)	$\sqrt{f'_c}$ (MPa)
AL1	7.28	0.23	1.29	0.33	7.3	0.23	1.30	—	—
AS1	8.5	0.27	1.51	0.40	6.86	0.22	1.22	9.3	1.66
MAS1	8.0	0.25	1.42	0.46	6.96	0.22	1.25	9.4	1.68
AL2	6.26	0.2	1.06	0.31	5.1	0.16	0.87	—	—
AS2	6.34	0.2	1.06	0.42	4.92	0.16	0.82	6.75	1.13

9.4 Summary of the Investigations on Interior Wide Beam-Column Joints

Table 9-3 shows the results of the interior wide beam-column joints from the experiments and analyses. The load versus displacement histories show that the specimens had adequate load-carrying capacity without strength degradation up to a drift level of 4% or more, and with little degradation during the last cycle. This behaviour is an indication of adequate or satisfactory performance of these tested specimens. The level of drift at which the nominal capacities were attained was around 1% drift. The lower stiffness or flexibility of the test specimens was somewhat due to the lower stiffness of the shallow wide-beam as compared to normal depth beam. Among these specimens, the bond deterioration was reflected in the load vs. horizontal displacement plots, in which the hysteretic loops pinching were observed due to the reduction in the specimen stiffness at low load levels.

Due to a larger section of the wide beam, the tension stresses in the beam transverse reinforcement were much lower. Therefore, the requirement of beam shear reinforcement can be relaxed.

Table 9-3 Summary of the experimental and analytical results of the interior wide-beam-column joints

Test Specimen	Column-to-Beam moment strength ratio	Theoretical capacity	Drift ratio at first yield	Maximum capacity in the test (kN)	Analytical capacity from FEM analysis (kN)
IWB-1	>4	±249	1.0%	+277 / -268	+308 / -286
IWB-2	1.5	±207	1.5%	+215 / -207	+220 / -210
IWB-3	>4	±244	1.0%	+256 / -248	+256 / -248

Based on the results of these tests and previous tests conducted by Hatamoto *et al* [H4], it was found that the effective beam width cannot reach the actual beam width when the beam to column width ratio is higher than 3.

For Specimens IWB1 and IWB3, the seismic performance is controlled by the torsion strength of transverse beams. As the torsion moment demand increase, the hysteretic loops will slip associated with the decreases of hysteresis energy dissipation capacity. In some cases, the specimen may lose its load-carrying capacity after the transverse beam has lost its torsion capacity. Therefore, the transverse beam is a critical issue in a wide-beam system design, which needs careful design and detailing.

9.5 Summary of the Investigations on Exterior Wide Beam-Column Joints

Table 9-4 shows the results of the exterior wide beam-column joints from the experiments and analyses. All the exterior specimens were tested to a storey drift ratio of 4% in either loading direction and experienced displacement ductility factors of approximately 3.5 to 4. In Specimens EWB-1 and EWB-3, the obtained positive displacement ductility without losing load-carrying capacity was about 2.

Comparing the test results of Specimen EWB-2 with those of Specimen EWB-1 or EWB-3, it is noticed that a lower beam to column width ratio in Specimen EWB-1 or EWB-3 did not result in a noticeable improvement of the response. In contrast, Specimen EWB-2 performed well, even though the bar diameter to anchorage depth ratio did not satisfy the design recommendations. This implies that the performance of an exterior wide beam-column connection can be improved when the beam bar anchorage ratio is increased. The inadequate performance of Specimens EWB-1 and EWB-3 is mainly due to the cracking of the transverse beams, which indicated that the seismic performance of an

exterior beam-column joint is much sensitive to the torsion strength of the transverse beams.

Figure 9-4 Summary of the experimental and analytical results of the exterior wide-beam-column joints

Test specimen	Column-to-Beam moment strength ratio	Theoretical capacity (kN)	Drift ratio at first yield	Maximum capacity in the test (kN)	Maximum capacity from FEM analysis (kN)
EWB-1	>4	+170.5 / -123.6	1.0%	+156 / -132.4	+158 / -152
EWB-2	1.5/0.68	+144.4 / -103	0.8%	+185 / -138.8	+169 / -125
EWB-3	>4	+170 / -123.6	1.0%	+160.9 / -120.3	—

9.6 Summary of the Parametric Study on the Beam-Column Joints

The concerns regarding the influence of parameters on the behaviour of conventional beam-column connections focus on the effect of column axial loads, beam longitudinal reinforcing ratios and presence of slabs. On the other hand, the parameters that have a significant influence on the behaviour of wide beam-column connections include column axial loads; beam to column width ratios; beam longitudinal bar anchorage ratios and the effect of transverse beams were investigated. The ranges of the primary parameters considered in this study are summary in **Table 9-5**.

9.6.1 Column Axial Load

In Chapter 8, variables influence on the joint shear behaviour is investigated through parametric studies. It was found that both the conventional beam-column joints and wide beam-column joints are sensitive to the column axial loads. A column axial load with the magnitude from $0.1 A_g f'_c$ to $0.2 A_g f'_c$ will evidently increase the joint shear resistance;

however, the influence is limited when the magnitude exceeds $0.2 A_g f'_c$. Thus, $0.2 A_g f'_c$ is a reasonable limit in obtaining a desired performance. It was found during the previous experimental investigations that bond condition was not assured using the bond parameter limits proposed by design codes. The increased flexibility of wide beam-column joints due to the deterioration of bond could create problems but the presence of an axial load should cause a reduction in the pinching of the hysteretic loops.

Table 9-5 Summary of the investigated parameters for conventional beam-column and wide beam-column joint's models

Models	Description	Ranges investigated
Conventional beam-column joint's models	Column axial load, N_c	0 to $0.4 A_g f'_c$
	Beam longitudinal reinforcing ratio, A_s / A'_s	0.75:1, 1:1, 1:2
	Slab width, $b_b + 2kh_b$	$k = 0$ to 4
Wide beam-column joint's models	Column axial load, N_c	0 to $0.6 A_g f'_c$
	Beam width to column width ratio, b_w / b_c	2.0 to 4.0
	Beam longitudinal bar anchorage ratio, R_{anchor}	33% to 75%
	Concrete compressive strength, f'_c	C20 to C70

9.6.2 Effective Slab Width and Beam-to-Column Width Ratio

The parametric study presented in Chapter 8 and the experimental study carried out in Chapter 5 showed that, for a conventional beam-column joint, the effective slab width cannot exceed the beam width plus two times the beam depth on each side of the beam.

In a wide beam-column joint, increase of the beam width will be associated with the change of the number of beam bars. Thus, not only the width ratio was increased but other design variables were also changed, for example the column-to-beam moment strength ratio, the

joint shear and the percentage of bars anchored in the column core. If the increase in the beam to column width ratio has been accompanied by suitable modifications in the other design variables, the specimen will possess enough strength and stiffness to withstand large interstory drifts without collapse or significant strength deterioration and the lag in the spreading of the plastic hinges will be reduced. If all the design parameters are kept close to the limits proposed by ACI Committee 352 but the special measures are not taken to prevent or delay the deterioration of bond, the specimens will still experience bond deterioration that lead to the pinching of the hysteretic loops, which is an indication of the softening of the connections. It is suggested that the width of wide beam should be limited to 3.0 for the interior joints and 2.0 for the exterior joints both in terms of beam-to-column width ratio.

9.6.3 Development of Wide-Beam Plastic Hinges

It is very important for a beam to develop a full plastic hinge within allowed drift levels as it guarantees the assumed design strength to be achieved before a collapse mechanism is developed in the event of a severe earthquake. However, there can be a significant delay in the development of a full-width plastic hinge in a wide-beam, which is more depend on the bond conditions of the bars and on the confinement of the outside region of the joint than it is on the beam width.

In the current experimental and analytical investigations, both interior and exterior wide-beam connections are not able to develop full-width plastic hinges at both side of the column. By enhancing the bond conditions of the bars passing outside the column core and by improving the confinement of the concrete in the outside region of the joint, the delay in the development of the plastic hinges might be minimized. Therefore, bond behaviour is an important factor for both conventional beam-column joints and wide beam-column connections, which required further studies to be carried out.

9.7 Recommendations for Future Work

Up to now, the research data on lightly reinforced beam-column joints and wide beam-column joints is insufficient to provide a better understanding of the performance of such kind of structures. Therefore, it is necessary to conduct more investigations through both experimental and analytical approaches.

- (1) Lap splices effect was not investigated in this study. As a typical reinforcing technology in the early design and construction practice, this should be a concern in further research.
- (2) Due to a lack of confinement, the bond between the reinforcement and the surrounding concrete is very important for a beam-column joint. It was shown in this study that the ACI 318 [A2] provisions may not be adequate in preventing the bond deterioration occurring. Using NZS 3101 [C1] recommendation in the design of beam-column joints was suggested but this needs verification through further investigations.
- (3) The torsion strength of the transverse beam interacted with the flexural strength ratio significantly affect the determination of the effective slab width. Therefore, in the future, the relationship between the effective slab width and the torsion strength as well as the column to beam flexural strength ratio should be investigated. Further research should also quantify the effects of transverse beam cracking on straight bar anchorage.
- (4) In this study, testes on both the lightly reinforced beam-column joints and the wide beam-column joint were carried out without applying of column axial loads. Future studies should investigate the seismic performance of beam-column joints with consideration of column axial load. In addition, it should also address the effect of axial load on straight bar anchorage.

(5) The specimen story drift imposed on the isolated subassemblage does not correspond one-to-one with expected whole-building story drift. This may occur for a numbers of reasons:

- (i) Forces can redistribute in a frame, but not in a subassemblage.
- (ii) The fixtures used to test isolated subassemblages may contain flexibility or other areas that soften the observed story shear versus displacement relationship.
- (iii) Whole-building models allow the building to be bent in shapes with inflection points at the location other than the mid-height of columns and mid-span of beams.
- (iv) The subassemblage may not be tested under axial loads.

This is a question that is raised in the research, which should apply to all reinforced concrete frames, both wide beam and normal beam. Further studies are required with concern of the relationship between specimen story drift and the drift of the overall building.

- (6) Further studies are required for the use of either deep transverse beams or very deep columns, which will influence the transfer of forces from the transverse beam to the column.
- (7) The finite element analyses in this research were conducted based on the current modelling criteria and analytical technology, which could not precisely evaluate the performance of RC frame buildings and components. More efforts have to be made to enhance the finite element analysis technology. To achieve this purpose, extensive experimental investigations are required in the future.

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