Experimental and analytical studies on the seismic behavior of reinforced concrete columns with light transverse reinforcement

Tran, Cao Thanh Ngoc

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**NANYANG
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UNIVERSITY**

**EXPERIMENTAL AND ANALYTICAL STUDIES
ON THE SEISMIC BEHAVIOR OF REINFORCED
CONCRETE COLUMNS WITH LIGHT TRANSVERSE
REINFORCEMENT**

**TRAN CAO THANH NGOC
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
2010**

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ON THE SEISMIC BEHAVIOR OF REINFORCED
CONCRETE COLUMNS WITH LIGHT TRANSVERSE
REINFORCEMENT**

TRAN CAO THANH NGOC

School of Civil and Environmental Engineering

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ABSTRACT

Structures made up of reinforced concrete columns with light transverse reinforcement are very common in a region of low to moderate seismicity, and are the predominant structural system in Singapore. Recent post-earthquake investigations have indicated that extensive damage in reinforced concrete columns with light transverse reinforcement occurs due to excessive shear deformation that subsequently leads to shear failure, axial failure and eventually full collapse of the structures. Therefore, a thorough evaluation of reinforced concrete columns with light transverse reinforcement is needed to understand the seismic behavior of these structures.

For this purpose, an experimental program carried out on reinforced concrete columns with light transverse reinforcement subjected to seismic loading is conducted. Ten 1/2-scale reinforced concrete columns with light transverse reinforcement are tested to investigate the seismic behavior of these columns. The variables in the test specimens include column axial loads, aspect ratios, and cross sectional shapes. The specimens are tested to the point of axial failure under a combination of a constant axial load and quasi-static cyclic loadings to simulate earthquake actions. Experimental results obtained include hysteretic responses, cracking patterns, strains in reinforcing bars, displacement decomposition and cumulative energy dissipation.

Next, an analytical approach, coupling flexure and shear deformations, is proposed to evaluate the initial stiffness of reinforced concrete columns subjected to seismic loading. A comprehensive parametric study is carried out based on the proposed approach to investigate the influences of several critical parameters. A simple equation is then proposed to estimate the initial stiffness of reinforced concrete columns. The applicability and accuracy of the proposed approach and equation are verified with the experimental data obtained from the current experimental program and studies in the literature.

Finally, a theoretical model is developed to estimate the displacement at axial failure of reinforced concrete columns with light transverse reinforcement subjected to seismic loads. The model is calibrated with the data obtained from testing the actual reinforced concrete columns up to the point of axial failure in studies in the literature. The applicability and accuracy of the proposed model are then verified with the test results obtained from the current experimental study.

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LIST OF SYMBOLS

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f'_c	Compressive strength of concrete
f_{yt}	Yield strength of transverse reinforcement
f_{yl}	Yield strength of longitudinal reinforcement
b	Width of columns
h	Depth of columns
d	Distance from the extreme compression fiber to centroid of tension reinforcement
L	Clear height of columns
P	Applied column axial load
s	Spacing of transverse reinforcement
A_{st}	Total transverse reinforcement area within spacing s
n_{bars}	Number of longitudinal reinforcing bars
d_b	Diameter of longitudinal reinforcing bars
A_g	Cross section of columns
Δ_a	Displacement at axial failure

CHAPTER 3

f_y	Yield strength of reinforcing bars
f_u	Ultimate strength of reinforcing bars
ρ_v	Transverse reinforcement ratio ($\rho_v = A_v / sh$)
ρ_l	Longitudinal reinforcement ratio
ϵ_y	Yield strain of reinforcing bars
f'_c	Compressive strength of concrete

M_u	Theoretical flexural moment of columns
V_u	Theoretical flexural strength of columns
M_y	Theoretical yield moment of columns
V_y	Theoretical yield force of columns
V_{cr}	Cracking shear force of columns
V_n	Nominal shear strength of columns
A_g	Cross section of columns
f_{yt}	Yield strength of transverse reinforcement
d	Distance from the extreme compression fiber to centroid of tension reinforcement
s	Spacing of transverse reinforcement
A_v	Total transverse reinforcement area within spacing s
a/d	Aspect ratio
P	Applied column axial load
θ_{f2}	Rotation of segment 2 due to flexure
δ_{2L}	Displacement measured by the left transducer at segment 2
δ_{2R}	Displacement measured by the right transducer at segment 2
h_t	Distance between the transducers
ϕ_2	Average curvature at segment 2
S_2	Depth of segment 2
δ_{f2}	Horizontal deflection of columns due to the flexural rotation of segment 2
x_{f2}	Distance from the center of the column to the center of segment 2
δ_f	Total horizontal deflection of columns due to the flexural rotations
γ_{s1}	Average shear distortion at segment 1
$\delta_{s1}, \delta'_{s1}$	Changes in length of the diagonal
L_{s1}	Initial length of the diagonals
α_{s1}	Angle between the diagonals and the vertical

δ_{s1}	Horizontal deflection of column due to the shear distortion of segment 1
x_{s1}	Vertical distance of the region in estimating the average shear distortion
δ_s	Total horizontal deflection due to shear distortions

CHAPTER 4

DR	Drift Ratio
PL	Performance Level
a / h	Aspect ratio
$V_{\max}$	Maximum shear force of columns
V_n	Nominal shear strength of columns
V_{cr}	Cracking shear force of columns
V_y	Theoretical yield force of columns
V_u	Theoretical flexural strength of columns
A_g	Area of cross section of columns
f'_c	Compressive strength of concrete

CHAPTER 5

A_g	Area of cross section of columns
I_g	Moment of inertia of gross section
E_c	Concrete elastic modulus
f'_c	Compressive strength of concrete
f_{yt}	Yield strength of transverse reinforcement
P	Applied column axial load
b	Width of columns
h	Depth of columns
d	Distance from the extreme compression fiber to centroid of tension reinforcement

L	Clear height of columns
A_v	Total transverse reinforcement area within spacing s
s	Spacing of transverse reinforcement
a / d	Aspect ratio
ρ_l	Longitudinal reinforcement ratio
$V_{\max}$	Maximum shear force of columns
V_n	Nominal shear strength of columns

CHAPTER 6

f'_c	Compressive strength of concrete
I_e	Effective moment of inertia
I_g	Moment of inertia of gross section
V_u	Theoretical flexural strength of columns
M_y	Theoretical yield moment of columns
V_y	Theoretical yield force of columns
V_{cr}	Cracking shear force of columns
$V_{\max}$	Maximum shear force of columns
Δ'_y	Displacement at yield force
Δ'_{flex}	Displacement due to flexure and bar slip at yield force
Δ'_{shear}	Displacement due to shear at yield force
ϕ'_y	Curvature at yield force
L	Clear height of columns
L_{sp}	Strain penetration length
f_{yl}	Yield strength of longitudinal reinforcing bars
d_b	Diameter of longitudinal reinforcing bars
$\varepsilon_{y,CL}$	Axial strains at the center of columns

$\varepsilon_{y,top}$	Axial strain at the extreme tension fiber
$\varepsilon_{y,bot}$	Axial strain at the extreme compression fiber
f_{sy}	Stress in transverse reinforcing bars at yield force
d	Distance from the extreme compression fiber to centroid of tension reinforcement
s	Spacing of transverse reinforcement
A_v	Total transverse steel area within spacing s
θ	Angle of diagonal compression strut
ε_x	Strain in transverse reinforcing bars at yield force
ε_{yt}	Yield strain of transverse reinforcing bars
E_s	Elastic modulus of steel
b	Width of columns
L_{cs}	Effective depth of the diagonal strut
a/h	Aspect ratio
ε_2	Compressive strain in the concrete compression strut
ε_1	Tensile strain in the concrete compression strut
E_c	Elastic modulus of concrete
f_{ce}	Effective compressive strength of concrete
γ_{xy}	Shear strain
γ_{xy}^i	Shear strain at lower section of segment i
γ_{xy}^{i+1}	Shear strain at upper section of segment i
h_i	Height of segment i
n	Number of segments
K_i	Initial stiffness of columns
k	Stiffness ratio
K_{i-exp}	Experimental initial stiffness of columns
K_{i-p}	Proposed initial stiffness of columns
K_{i-ACI}	Initial stiffness of columns calculated based on ACI 318-08

K_{i-ASCE}	Initial stiffness of columns calculated based on ASCE 42
K_{i-FEMA}	Initial stiffness of columns calculated based on FEMA
K_{i-PP}	Initial stiffness of columns calculated based on Paulay and Priestley
K_{i-EE}	Initial stiffness of columns calculated based on Elwood and Eberhard
A_g	Area of cross section of columns
$R_a, a/d$	Aspect ratios
R_n	Axial load ratio
ρ_v	Transverse reinforcement ratio
ρ_l	Longitudinal reinforcement ratio
f_{yl}	Yield strength of longitudinal reinforcing bars
P	Applied axial load
k_{exp}	Experimental stiffness ratio of columns
k_p	Proposed stiffness ratio of columns
k_{ACI}	Stiffness Ratio of columns calculated based on ACI 318-08
k_{ASCE}	Stiffness Ratio of columns calculated based on ASCE 42
k_{FEMA}	Stiffness Ratio of columns calculated based on FEMA
k_{PP}	Stiffness Ratio of columns calculated based on Paulay and Priestley
k_{EE}	Stiffness Ratio of columns calculated based on Elwood and Eberhard

CHAPTER 7

W_{ext}	External work
W_{int}	Internal work
P	Applied axial load
W_c	Internal work done by concrete
W_{sv}	Internal work done by transverse reinforcement
W_{sl}	Internal work done by longitudinal reinforcement

Δ_a^*	Horizontal displacement due to the sliding between cracking surfaces at the point of axial failure
Δ_{av}^*	Vertical displacement due to the sliding between cracking surfaces at the point of axial failure
ρ_l	Longitudinal reinforcement ratio
b	Width of columns
h	Depth of columns
f_{sl}	Axial strength of longitudinal reinforcement at axial failure
f_{yt}	Yield strength of transverse reinforcement
d	Distance from the extreme compression fiber to centroid of tension reinforcement
s	Spacing of transverse reinforcement
A_{st}	Total transverse reinforcement area within spacing s
θ	Angle of shear crack
V_c	Shear force carried by concrete
P_{sl}	Axial strength contributed by longitudinal reinforcement at the point of axial failure
P_{st}	Axial strength contributed by transverse reinforcement at the point of axial failure
P_c	Axial strength contributed by concrete at the point of axial failure
η_{sl}	Ratio of the axial strength of longitudinal reinforcing bars at axial failure to the yield strength of longitudinal reinforcement
A_g	Cross sectional area
k	Parameter depends on the displacement ductility demand
Δ_a	Horizontal displacement of columns at the point of axial failure
Δ_y	Yield displacement of columns
L_d	Damaged length
δ_a^*	Ratio of the horizontal displacement due to the sliding between cracking surfaces at axial failure to the damaged length

CHAPTER 1

INTRODUCTION

1.1 Problem Statement

A large number of existing reinforced concrete (RC) columns in zones of low to moderate seismicity has not been designed following the requirements of modern seismic design codes. These are generally termed as non-seismically detailed RC columns. Vital deficiencies in such columns include typical reinforcement details as (1) lightly, widely spaced and poorly anchored transverse reinforcement, and (2) lap-splice details. Recent post-earthquake investigations [E2, E3, E4, L1, M1, S3] have indicated that extensive damage in non-seismically detailed RC columns occurs due to excessive shear deformation that subsequently leads to shear failure, axial failure and eventually full collapse of the structures as shown in **Figure 1.1** and **1.2**. Therefore, a thorough evaluation of non-seismically detailed RC columns is needed to understand the seismic behavior of these structures.

Figure 1.1 Failures of Columns during 1999 Kocaeli Earthquake
(reprinted from Elwood *et al.* 2005 [E5])

Extensive experimental research studies have been conducted on ductile columns in different countries throughout past decades, which have given a better understanding on the seismic behavior of ductile columns. However, there are limited research studies related to non-seismically detailed RC columns. In addition, most tests of RC columns subjected to seismic loads have been terminated shortly after loss of lateral load resistance. Few tests on RC columns have been carried out to the point of axial failure. This has resulted in a limited understanding of the failure and collapse mechanisms governing non-seismically detailed structures.

Figure 1.2 Damaged Column during 1995 Kobe Earthquake
(reprinted from Yoshimura *et al.* 2003 [Y2])

Therefore a study is being undertaken at Nanyang Technological University (NTU), Singapore with an aim to attain a better understanding of the seismic behavior of non-seismically detailed RC columns. The present investigation is planned to carry out both experimental and analytical studies to provide further contribution to this field of research. The results will be useful in obtaining a better understanding of the failure and collapse mechanisms governing non-seismically detailed RC columns. It should be possible to improve the behavior of such columns during earthquakes by knowing the deficiencies.

1.2 Objectives and Scope

The research reported herein is concerned with the seismic behavior of RC columns with light transverse reinforcement. This research consists of the following experimental and analytical components:

1. Collecting, reviewing and interpreting data related to the seismic behavior of RC columns tested to the point of axial failure.
2. Conducting a series of tests involving ten RC columns with light transverse reinforcement to study their seismic behaviors to the point of axial failure.
3. Developing an analytical method to estimate the initial stiffness of RC columns.
4. Proposing a simple model to estimate the displacements at the point of axial failure of RC columns with light transverse reinforcement.

1.3 Report Organization

The report is organized into eight chapters starting with the introduction and objectives in this chapter. **Chapter 2** presents a literature review of previous experimental research studies on the subject of RC columns tested to the point of axial failure. **Chapter 3** describes details of the test specimens and the loading program. The test results are given in **Chapter 4**. **Chapter 5** discusses and compares the experimental results between test specimens. **Chapter 6** is dedicated to developing a simple equation to estimate the initial stiffness of RC columns subjected to seismic loadings. A simple model is developed in **Chapter 7** to estimate the displacements of RC columns with light transverse reinforcement at axial failure. **Chapter 8** summarizes the works done in this study and presents the main conclusions obtained from these experimental and analytical investigations. Suggestions for future works are also given in this chapter.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Extensive experimental studies in past decades have provided a fundamental understanding of the seismic behavior of reinforced concrete (RC) columns in many aspects. Current seismic design codes such as ACI 318-08 [A1], NZS 3101 [N2], AIJ guidelines [A3] and EC8 [E1] were established based on these studies. These seismic design codes require considerable amounts of transverse reinforcement to be placed in the plastic hinges of columns. However, existing RC columns in low to moderate seismic hazard zones such as Malaysia and Singapore were constructed with light and widely spaced transverse reinforcement. The current codes do not provide the necessary information to assess the strength and deformation capacity for these non-seismically detailed columns. In addition, most tests of RC columns subjected to seismic loading have been terminated shortly after loss of lateral load resistance. Only few experimental research studies on the seismic performance of RC columns conducted in Japan, Singapore and USA were carried out to the point of axial failure. This chapter reviews these experimental research studies.

2.2 Previous Experimental Studies on the Seismic Behavior of RC Columns Tested to the Point of Axial Failure

2.2.1 Research Conducted by Yoshimura [Y1]

Yoshimura *et al.* [Y1] conducted two series of tests to study the axial failure phenomenon of large-scale cantilever RC columns. Reinforcement details of the test specimens were shown in **Figure. 2.1**. The first series (FS series) consisting of specimens with an aspect ratio of 2.00 was designed to reach flexural yielding before shear failure, whereas the specimens in the second series (S series) with an aspect ratio of 1.50 were designed to fail in shear prior to reaching flexural yielding.

Three loading schemes, namely monotonic, unidirectional cyclic and bidirectional cyclic loading were investigated in this study. Moderate column axial loads of $0.26 f_c' A_g$ and $0.20 f_c' A_g$ were applied to the FS and S series, respectively. Details of each test specimen and used materials' properties were tabulated in **Table 2.1**.

Figure 2.1 Reinforcement Details of Specimens Tested by Yoshimura *et al.* [Y1]

It was found that the lateral and vertical displacements at the ultimate limit state (axial failure) varied depending on loading history applied to the specimens. The specimens subjected to bilateral loads obtained the lowest ultimate lateral displacements in both series. The obtained displacements at the ultimate limit state of all test specimens were tabulated in **Table 2.1**.

2.2.2 Research Conducted by Lynn [L2]

Lynn *et al.* [L2] carried out tests on eight full-scale RC columns with light transverse reinforcement subjected to low or moderate column axial loads. The test columns had typical details of those built before the mid-1970s, including light transverse reinforcement and lap-splices at the bottom of the column as shown in **Figure 2.2**. The variables in the test specimens included percentages of longitudinal

and transverse reinforcement, lap-splice details and column axial loads. Details of each test specimen were tabulated in **Table 2.1**.

Figure 2.2 Typical Reinforcement Details of Specimens Tested by Lynn *et al.* [L2] and Sezen *et al.* [S1]

The following conclusions were derived by Lynn *et al.* [L2] based on the test results:

- Longitudinal reinforcement lap-splices having length equal to 20 times the longitudinal bar diameter were adequate to develop yield stress in longitudinal reinforcing bars.
- Specimens that reached the flexural strength before the shear strength exhibited more ductile response.
- Axial failure occurred at or after significant loss of lateral load resistance.

2.2.3 Research Conducted by Sezen [S1]

Sezen *et al.* [S1] conducted tests on four identical full-scale RC columns subjected to either cyclic or monotonic lateral loads. The test specimens had similar material properties and details with Lynn *et al.*'s specimens [L2] as shown in **Figure 2.2**. The specimens were tested under unidirectional lateral loads with either constant or varying column axial loads. Based on the experimental results, Sezen *et al.* [S1] concluded that:

- The responses of RC columns with identical properties varied considerably with the magnitude and history of axial and lateral loads.
- Axial failure did not occur in the specimen with a low column axial load until the applied displacements had increased substantially beyond shear failure.
- The column with a high column axial load exhibited a brittle shear failure phenomenon. And axial failure occurred immediately after shear failure.

2.2.4 Research Conducted by Nakamura [N1]

Figure 2.3 Reinforcement Details of Specimens Tested by Nakamura *et al.* [N1]

An experimental program consisting of four identical columns as shown in **Figure 2.3** was conducted by Nakamura *et al* [N1]. The variables in the test specimens were column axial loads and lateral loading schemes. Either monotonic or cyclic lateral loads were applied to the test specimens. Two column axial loads of $0.18 f'_c A_g$ and $0.27 f'_c A_g$ were exerted to the columns. The specimens were tested to the point of axial failure.

Nakamura *et al.* [N1] found that the magnitude of axial loads and the history of lateral loads significantly affected the seismic behavior of the test specimens, which were similar to Sezen *et al.*'s findings [S1].

2.2.5 Research Conducted by Yoshimura [Y2]

Six short RC columns with an aspect ratio of 1.00 as shown in **Figure 2.4** were tested by Yoshimura *et al* [Y2]. The effects of column axial loads, percentages of longitudinal reinforcement and loading history on displacements at axial failure were studied in Yoshimura *et al.*'s experimental program [Y2]. Through experimental results, Yoshimura *et al.* [Y2] concluded that a smaller displacement at axial failure was obtained in the specimens with a lower percentage of longitudinal reinforcement.

Figure 2.4 Typical Reinforcement Details of Specimens Tested by Yoshimura *et al.* [Y2]

2.2.6 Research Conducted by Yoshimura [Y3]

Yoshimura *et al.* [Y3] carried out tests on eight 1/2-scale RC columns. The specimens were detailed to display either shear failure or shear failure after flexural yielding under cyclic loading. The variables in the test specimens included percentages of longitudinal and transverse reinforcement, and column axial loads. All test specimens had an aspect ratio of 2.00. Typical details of the test specimens are shown in **Figure 2.5**.

Figure 2.5 Typical Reinforcement Details of Specimens Tested by Yoshimura *et al.* [Y3]

The experimental results showed that:

- Axial failure occurred in the specimens with shear failure mode (without prior flexural yielding) when the shear resistance degraded to nearly zero.
- Specimens with flexural-shear failure mode collapsed simultaneously with onset of shear failure.

2.2.7 Research Conducted by Ousalem [O3]

Ousalem *et al.* [O3] conducted two series of test on scaled RC columns. The test specimens had similar details with Nakamura *et al.*'s specimens [N1]. In the first series, the effects of column axial loads were investigated; whereas the influences of lateral loading history on the displacements at axial failure were studied in the second series. The experimental results in the two series showed that:

- The lateral displacement capacity and shear strength degradation were influenced by the column axial load. A higher axial load resulted in a steeper failure plan and a lower lateral displacement capacity.
- The lateral displacement capacity of the specimens with a low transverse reinforcement ratio was not influenced by the lateral loading history.

2.2.8 Research Conducted by Tran [T1]

Figure 2.6 Reinforcement Details of Specimen Tested by Tran *et al.* [T1]

Tran *et al.* [T1] tested one RC column with light transverse reinforcement as shown in **Figure 2.6**. The column was tested to the point of axial failure under a combination of a constant column axial load of $0.30 f'_c A_g$ and quasi-static cyclic

loadings to simulate earthquake actions. The test result showed that axial failure occurred in the test specimen at a drift ratio of 1.82% immediately after shear failure due to the fracture of transverse reinforcements and buckling of longitudinal reinforcements.

2.3 Conclusions Drawn From the Previous Experimental Studies

A database consisting of 48 RC columns tested to the point of axial failure had been collected as tabulated in **Table 2.1**. The database contains the displacement at axial failure, geometry, axial load and material properties of the collected specimens.

Reviewing the previous studies (Yoshimura *et al.* [Y1]; Lynn *et al.* [L2]; Sezen *et al.* [S1]; Nakamura *et al.* [N1]; Yoshimura *et al.* [Y2]; Yoshimura *et al.* [Y3]; Ousalem *et al.* [O1]; and Tran *et al.* [T1]) in the area of RC columns tested to the point of axial failure, the following problem was identified:

1. All test columns had a square cross sectional shape. The seismic behavior of columns with other cross sectional shapes tested to the point of axial failure has not been studied.
2. The effects of column axial loads on the displacement at axial failure had not been studied in detail. There were only two column axial loads investigated in each experimental program.
3. Most experimental studies concentrated on RC columns with an aspect ratio of either less than 1.50 or larger than 3.00.

Table 2.1 Database of RC Columns Tested to the Point of Axial Failure

Specimen	Column Section						Transverse Reinforcement			Longitudinal Reinforcement			Δ_a (mm)
	b (mm)	h (mm)	d (mm)	L (mm)	f_c' (MPa)	P (kN)	s (mm)	A_v (mm ²)	f_{yt} (MPa)	n_{bars}	d_b (mm)	f_{yl} (MPa)	
Yoshimura <i>et al.</i> [Y1] *													
FS0	300	300	255	600	27.0	632	75	157	355	12	19.0	387	54.6
FS0	300	300	255	600	27.0	632	75	157	355	12	19.0	387	50.4
FS0	300	300	255	600	27.0	632	75	157	355	12	19.0	387	31.8
S1	400	400	350	600	27.0	803	180	157	355	16	22.0	547	51.6
S2A	400	400	350	600	27.0	803	180	157	355	16	22.0	547	52.8
S2A	400	400	350	600	27.0	803	180	157	355	16	22.0	547	40.2
Lynn <i>et al.</i> [L2] **													
3CLH18	457	457	393	2946	25.6	503	457	142	400	8	32.3	335	61.0
2CLH18	457	457	397	2946	33.1	503	457	142	400	8	25.4	335	91.3
3SLH18	457	457	393	2946	25.6	503	457	142	400	8	32.3	335	91.3
2SLH18	457	457	397	2946	33.1	503	457	142	400	8	25.4	335	106.6
2CMH18	457	457	397	2946	25.7	1512	457	142	400	8	25.4	335	30.3
3CMH18	457	457	393	2946	27.6	1512	457	142	400	8	32.3	335	61.0
3CMD12	457	457	393	2946	27.6	1512	305	245	400	8	32.3	335	61.0
3SMD12	457	457	393	2946	25.6	1512	305	245	400	8	32.3	335	61.0

Sezen <i>et al.</i> [S1] **													
2CLD12	457	457	392	2946	21.1	667	305	245	469	8	28.7	469	146.0
2CHD12	457	457	392	2946	21.1	2669	305	245	469	8	28.7	469	55.0
2CVD12	457	457	392	2946	20.9	1491	305	245	469	8	28.7	469	86.0
2CLD12M	457	457	392	2946	21.1	667	305	245	469	8	28.7	469	161.0
Nakumura <i>et al.</i> [N1] **													
N18M	300	300	255	900	26.5	429	100	56.5	375	12	16.0	380	92.7
N18C	300	300	255	900	26.5	429	100	56.5	375	12	16.0	380	185.4
N18C	300	300	255	900	26.5	876	100	56.5	375	12	16.0	380	42.3
N18C	300	300	255	900	26.5	876	100	56.5	375	12	16.0	380	27.0
Yoshimura <i>et al.</i> [Y2] **													
2M	300	300	255	600	25.2	430	100	56.5	392	12	16.0	396	67.2
2C	300	300	255	600	25.2	430	100	56.5	392	12	16.0	396	46.8
3M	300	300	255	600	25.2	657	100	56.5	392	12	16.0	396	33.6
3C	300	300	255	600	25.2	657	100	56.5	392	12	16.0	396	31.8
2M13	300	300	255	600	25.2	430	100	56.5	392	12	13.0	350	24.6
2C13	300	300	255	600	25.2	430	100	56.5	392	12	13.0	350	18.0
Yoshimura <i>et al.</i> [Y3] **													
No.1	300	300	255	1200	30.7	552	100	56.5	392	12	16.0	402	160.8
No.2	300	300	255	1200	30.7	552	150	56.5	392	12	16.0	402	64.8
No.3	300	300	255	1200	30.7	552	200	56.5	392	12	16.0	402	24.0

No.4	300	300	255	1200	30.7	828	100	56.5	392	12	16.0	402	24.0
No.5	300	300	255	1200	30.7	967	100	56.5	392	12	16.0	402	24.0
No.6	300	300	255	1200	30.7	552	100	56.5	392	12	13.0	409	63.6
No.7	300	300	255	1200	30.7	552	150	56.5	392	12	13.0	409	24.0
Ousalem <i>et al.</i> (2002) [O3] **													
C1	300	300	260	900	13.0	364	160	39.3	587	12	13.0	340	9.1
C4	300	300	260	900	13.0	364	75	56.5	384	12	13.0	340	36.2
C8	300	300	260	900	20.0	486	75	56.5	384	12	13.0	340	13.6
C12	300	300	260	900	20.0	324	75	56.5	384	12	13.0	340	72.6
D1	300	300	260	600	27.7	540	50	56.5	398	12	13.0	447	24.3
D16	300	300	260	600	26.1	540	50	56.5	398	12	13.0	447	24.0
D11	300	300	260	900	28.2	540	150	56.5	398	16	13.0	447	16.9
D12	300	300	260	900	28.2	540	150	56.5	398	16	13.0	447	17.5
D13	300	300	260	900	26.1	540	50	56.5	398	16	13.0	447	31.5
D14	300	300	260	900	26.1	540	50	56.5	398	16	13.0	447	90.5
D15	300	300	260	900	26.1	540	50	113	398	16	13.0	447	161.0
D5	300	300	260	750	28.5	540	50	56.5	398	12	16.0	431	46.1
Tran <i>et al.</i> [T1] **													
SC01	350	350	309	1700	49.3	1812	125	56.5	400	8	25	409	30.1

Note: * Single Curvature Specimens; ** Double Curvature Specimens.

CHAPTER 3

EXPERIMENTAL PREPARATION AND TEST PROCEDURE

3.1 Introduction

Structures made up of reinforced concrete (RC) columns with light transverse reinforcement are very common in a region of low to moderate seismicity, and are the predominant structural system in Singapore. Recent post-earthquake investigations have indicated that extensive damage in reinforced concrete columns with light transverse reinforcement occurs due to excessive shear deformation that subsequently leads to shear failure, axial failure and eventually full collapse of the structures. Therefore, a thorough evaluation of RC columns with light transverse reinforcement is needed to understand the seismic behavior of these structures. Furthermore, while reviewing the previous research studies (see **Chapter 2**); it was found that few experimental studies have been conducted on RC columns with light transverse reinforcement to the point of axial failure. Thus, it is necessary to carry out additional experimental investigations to provide more information and further understanding of failure and collapse mechanisms of such structures.

For this purpose, an experimental program carried out on RC columns with light transverse reinforcement subjected to seismic loading was conducted. This experimental program consists of ten 1/2-scale RC columns with light transverse reinforcement. The variables in the test specimens include column axial loads, aspect ratios and cross sectional shapes. The specimens were tested to the point of axial failure under a combination of a constant axial load and quasi-static cyclic loadings to simulate earthquake actions. This chapter describes the details of the test specimens, the preparatory works and test procedures of the experimental program. The instrumentations used for load and displacement measurements are also described in details.

3.2 Test Setup

(a) Typical Details in Drawing

(b) Typical Details in Photograph

Figure 3.1 Experimental Setup

A schematic of the loading apparatus is shown in **Figure 3.1**. The column axial load was applied to the specimens using two actuators, each with a 1000 kN capacity through a L-shaped steel frame. The actuators were pinned at both ends to allow rotation during the test. The bottom and top bases of the specimen were fixed to a

reaction slab with four post-tensioned bolts and the L-shaped steel frame, respectively. Reversible horizontal displacements were applied to the specimen through an actuator with a 1000 kN capacity whose axis passed through the mid-height of the specimen, thus generating a double-bending loading condition to the specimen. This horizontal actuator was mounted to a reaction wall and the L-shaped steel frame as shown in **Figure 3.1**.

3.3 Description of Test Specimens

3.3.1 Details of Test Specimens

Table 3.1 Summary of Test Specimens

Specimen	Longitudinal Reinforcement	Transverse Reinforcement	f_c' (MPa)	$b \times h$ (mm×mm)	L (mm)	$\frac{P}{f_c' A_g}$
SC-2.4-0.20	8-T20 $\rho_l = 2.05\%$	2-R6 @ 125 $\rho_v = 0.13\%$	25.0	350 × 350	1700	0.20
SC-2.4-0.50						0.50
SC-1.7-0.05					1200	0.05
SC-1.7-0.20						0.20
SC-1.7-0.35						0.35
SC-1.7-0.50						0.50
RC-1.7-0.05		2-R6 @ 125 $\rho_v = 0.18\%$		250 × 490	1700	0.05
RC-1.7-0.20						0.20
RC-1.7-0.35						0.35
RC-1.7-0.50						0.50

Ten 1/2-scale RC columns with light transverse reinforcement were tested to investigate the seismic behavior of these columns. The variables in the test specimens as tabulated in **Table 3.1** include column axial loads, aspect ratios and cross sectional shapes. The longitudinal reinforcement in all test specimens consisted of 8-T20 bars (20 mm diameter). This resulted in a ratio of longitudinal reinforcement area to the gross sectional area of column to be 2.05%. The transverse reinforcement consisted of R6 bars (6 mm diameter) with 135° bent spaced at 125 mm. Deformed bars (T20) were used for the longitudinal

reinforcement while mild-steel bars (R6) were used for the transverse reinforcement. Details on the material properties of the steels are presented in the next section.

Figure 3.2 Reinforcement Details of Test Specimens (in mm)

Material Properties

In order to achieve more reliable results, the properties of materials used were determined through various tests. It is to be noted that the actual strength of the material supplied deviate from that of the specifications.

Reinforcing Steel Bars

The steel bars used in all specimens were the hot-rolled type. The longitudinal reinforcing bars were high yield strength deformed bars with a characteristic yield strength of 460 N/mm². Mild yield strength round bars with a characteristic yield strength of 250 N/mm² were used for the transverse reinforcement. Tensile tests were carried out on sample reinforcing bars to determine their true mechanical tensile properties. **Table 3.2** tabulates the mechanical tensile properties of the steel used.

Table 3.2 Measured Properties of Reinforcing Steel

Type	Grade (MPa)	Yield Strength, f_y (MPa)	Yield Strain, ϵ_y ($\times 10^{-6}$)	Ultimate Strength, f_u (MPa)
R6	250	392.6	2316.0	579.7
T20	460	408.0	2045.0	606.6

Note: R6 = Plain round bar of 6 mm diameter

T20 = Deformed high strength bar of 20 mm diameter

Concrete

Ready-mix concrete was used to cast the test specimens. The specified concrete compressive strength at 28 days was 25 MPa with maximum specified aggregate size of 10 mm for all specimens. Three 150 mm diameter by 300 mm high concrete cylinders were used to determine the actual compressive strength of the concrete for each specimen prior to each test. The average cylinder compressive strengths f'_c taken from three compression tests on the days of testing for each specimen were

summarised in **Table 3.3**. It is to be noted that the concrete cylinders were cast together with the specimens using the same batch of concrete mix and were cured under the same conditions. A good representation on the concrete strength of the specimens can therefore be achieved.

Table 3.3 Compressive Strength of Concrete

Specimen	Age at Test (days)	Average Compressive Strength f'_c (MPa)
SC-2.4-0.20	21	22.6
SC-2.4-0.50	25	24.2
SC-1.7-0.05	60	29.8
SC-1.7-0.20	40	27.5
SC-1.7-0.35	30	25.5
SC-1.7-0.50	35	26.4
RC-1.7-0.05	66	32.5
RC-1.7-0.20	26	24.5
RC-1.7-0.35	39	27.1
RC-1.7-0.50	36	26.8

3.3.2 Construction Process

The construction of the test specimens was tendered to a construction company due to the space limitation in the laboratory. The construction process consisted of several stages, including reinforcing cages, formworks, strain gauging, and casting and curing of the specimens. Details of each stage will be provided in the following sections.

Reinforcing Cages

All the longitudinal reinforcements and transverse reinforcements were cut to length and bent by the construction company. The installation of the strain gauges only took place after the completion of the reinforcing cages. This eliminates possible damage to the gauges that could occur during the process of tying the reinforcing

cages. **Figure 3.3** shows a completed reinforcing cage of the test specimen.

Figure 3.3 Typical Reinforcing Cages

Formwork

Figure 3.4 Formworks with Reinforcing Cages

In the construction, combinations of steel and wooden formworks were used. Steel plates with a thickness of 20 mm were used as the base of the formworks while plywood sheets with stiffeners were used as the supporting formworks. Before the placement of the reinforcing cages, the surfaces of the formworks were oiled so that

the formworks can be easily removed and the surface of specimens will not be damaged. Before the casting process, 25 mm thick concrete spacer blocks were placed on the underside as well as the side faces of the transverse reinforcements to ensure a clear concrete cover of 25 mm was achieved. Lifting hooks were installed at the bottom base of the specimens to facilitate the lifting process. **Figure 3.4** shows the constructed formworks with reinforcing cages prior to casting.

Casting and Curing

The concrete was provided by a local commercial ready-mix plant with the specified 28 days compressive strength to be 25 MPa. Chipping aggregates with a maximum size of 10 mm were used in the mix to ensure better flow of the concrete due to the limited concrete cover spacing of 25 mm. Although casting of the specimens in the upright position to simulate the real situation was very much desirable, the workmanship and time required to ensure precision of the specimens were lacking. It was therefore decided to cast the specimens horizontally and in a single pour. After casting, all the specimens were cured for seven days with damp hessian fabrics. The specimens were then transported to the laboratory for the test set-up.

3.3.3 Nominal Capacities

Table 3.4 summarizes the nominal capacities of the test specimens. The theoretical flexural strengths (V_u) were estimated using the tested material properties and in accordance with the recommendation provided by FEMA 356 [F1].

The theoretical yield forces (V_y) were calculated based on the yield moments (M_y) at which the first yield occurs in the longitudinal reinforcement or the maximum compressive strain of concrete reaches 0.002 at a critical section of the column.

The cracking shear forces (V_{cr}) were determined based on Priestley *et al.*'s equation [P4] as follows:

$$V_{cr} = 0.29\sqrt{f'_c}(0.8A_g) \text{ (MPa)} \quad (3.1)$$

where f'_c is the compressive strength of concrete and A_g is the cross section of columns.

The nominal shear strengths (V_n) were calculated based on FEMA 356 recommendations [F1]:

$$V_n = \frac{A_v f_{yt} d}{s} + \left(\frac{0.5\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{0.5\sqrt{f'_c} A_g}} \right) 0.8A_g \text{ (MPa)} \quad (3.2)$$

where f_{yt} is the yield strength of transverse reinforcement; d is the distance from the extreme compression fiber to centroid of tension reinforcement; s is the spacing of transverse reinforcement; A_v is the total transverse reinforcement area within spacing s ; the aspect ratio a/d shall not be taken greater than 3 or less than 2 and P is the applied column axial load.

Table 3.4 Nominal Capacities of Test Specimens

Specimen	M_y (kNm)	M_u (kNm)	V_{cr} (kN)	V_y (kN)	V_u (kN)	V_n (kN)	$\frac{V_u}{V_n}$
SC-2.4-0.20	184.7	196.5	135.1	217.3	231.2	199.1	1.16
SC-2.4-0.50	175.1	220.2	139.8	206.0	259.1	258.5	1.00
SC-1.7-0.05	149.2	181.9	155.1	248.7	303.2	209.8	1.45
SC-1.7-0.20	198.3	212.0	149.0	330.5	353.3	281.0	1.26
SC-1.7-0.35	194.0	224.4	143.5	323.3	374.0	318.3	1.17
SC-1.7-0.50	186.3	225.1	146.0	310.5	375.2	366.7	1.02
RC-1.7-0.05	192.2	237.3	162.0	226.1	279.2	239.7	1.16
RC-1.7-0.20	246.5	266.7	140.7	290.0	313.8	297.8	1.05
RC-1.7-0.35	268.9	300.5	147.9	316.4	353.5	354.5	1.00
RC-1.7-0.50	257.1	286.9	147.1	302.5	337.5	394.9	0.85

3.4 Loading Sequence and Test Procedure

The column axial load was applied slowly to the specimens until the designated level was achieved. During each test, the column axial load was maintained by manually adjusting the vertical actuators after each load step.

Figure 3.5 Loading Procedure

The lateral load was applied cyclically through the horizontal actuator in a quasi-static fashion as shown in **Figure 3.1**. The loading procedure consisting of displacement-controlled steps is illustrated in **Figure 3.5**.

3.5 Instrumentations of the Test

The test specimens had been extensively installed or mounted with measuring devices both internally and externally. Among those measurements recorded were lateral loads and displacements imposed at the top of the column, shear and flexure deformations at the critical regions of the specimen and also the strains in the steel reinforcing bars.

3.5.1 Measurement of Loads

Three in-built load cells in the computer-controlled actuators as shown in **Figure 3.1** were used to measure the applied axial and lateral loads on the specimen. The load cells were connected to the test system through signal cables and the load readings were obtained through the Multi-Purpose Test (MPT) program provided by the actuator's supplier. The load cells were factory-calibrated.

3.5.2 Measurement of Lateral Displacements

The displacements at top of the column were measured during the test procedure by using a Linear Variable Differential Transducer (LVDT) attached to magnetic stands, which were mounted on steel frames. **Figure 3.6** shows the arrangement of the LVDT for lateral displacements measurement on the test specimen.

Figure 3.6 Typical Arrangements of LVDT for Lateral Displacements Measurement

3.5.3 Measurements of Shear and Flexure Deformations

A series of LVDTs were placed at various locations of the specimens to measure the shear and flexure deformations. The LVDTs were attached on 3 mm thick steel plates and were mounted onto 10 mm steel rods embedded in the concrete. Such measuring devices were not applied to the full span of the column, but only focused on locations where potential deformations were likely to occur.

Figure 3.7 Arrangements of LVDTs and Linear Potentiometers for Shear and Flexure Deformations Measurement (in mm)

The readings from pairs of LVDTs along the column were used to measure flexure deformations of the test specimens. Shear distortions were measured by using LVDTs arranged in a cross-like manner. It is to be noted that at near failure stages of the test, crashing and spalling of concrete induced false readings on the measuring devices. The affected LVDTs were then removed as it no longer served

any purpose and to prevent them from being damaged. The overall arrangement of the instrumentations is shown in **Figure 3.7**. Methods for calculation of shear and flexure deformations will be presented in the following sections.

3.5.4 Measurements of Strains in Reinforcing Bars

- Strain Gauge on Transverse Reinforcements
- Strain Gauge on Longitudinal Reinforcements

Figure 3.8 Locations of Strain Gauges (in mm)

Strain gauges were used to measure the local strains in the reinforcing steel bars. All strain gauges were of the KFG type with 5 mm-gauge length, 120 Ω resistance and nominal gauge factor of 2.08. Soldering to the terminals was not required because these strain gauges had been pre-attached with 10 m long of 3 parallel vinyl-insulated lead wires. Due to the limited amount of available strain gauges, only reinforcing bars at critical locations much similar to that of the external measuring devices were installed with gauges. Details on the preparation and installation of the strain gauges will not be presented in this report as such the

standard procedures were well documented in various technical reports. The locations of the strain gauges for each test specimen were shown in **Figure 3.8**.

3.6 Displacement Decomposition

3.6.1 Flexure Deformation

The flexural deformations of columns were estimated from the discrete rotations in each segment along columns, which were measured from the pairs of LVDTs. The derivation of the following equations was based on Bernoulli hypothesis, which states that plane sections remain plane after deformation.

With reference to **Figure 3.9**, the rotation of segment 2 due to flexure (θ_{f2}) is given by:

$$\theta_{f2} = \frac{\delta_{2R} - \delta_{2L}}{h_t} \quad (3.3)$$

where δ_{2L} and δ_{2R} are the displacement measured by the left and right transducer at segment 2, respectively; and h_t is the distance between the transducers at segment 2 as illustrated in **Figure 3.9**.

The corresponding average curvature (ϕ_2) can be derived by:

$$\phi_2 = \frac{\theta_{f2}}{S_2} \quad (3.4)$$

Where S_2 is the depth of segment 2 as illustrated in **Figure 3.9**.

Figure 3.9 Evaluation of Flexure Deformations

The horizontal deflection of columns due to the flexural rotation of this particular segment (δ_{f2}) is given by:

$$\delta_{f2} = \theta_{f2} \times x_{f2} \quad (3.5)$$

where x_{f2} is the distance from the center of the column to the center of segment 2 as illustrated in **Figure 3.9**.

The total horizontal deflection of columns due to the flexural rotations is equal to the summation of the individual segment deflection:

$$\delta_f = \sum \frac{\delta_{iR} - \delta_{iL}}{h_t} \times x_{fi} \quad (3.6)$$

where subscript ' i ' represents the particular segment under consideration

3.6.2 Shear Deformation

The shear deformations of columns were measured by using pairs of LVDTs arranged in a cross-liked manner as described in the previous section.

With reference to **Figure 3.10**, the average shear distortion at segment 1 (γ_{s1}) is evaluated as follows:

$$\gamma_{s1} = \frac{\delta_{s1} - \delta'_{s1}}{2L_{s1}} \left(\tan \alpha_{s1} + \frac{1}{\tan \alpha_{s1}} \right) \quad (3.7)$$

where δ_{s1} and δ'_{s1} are the changes in length of the diagonals; L_{s1} is the initial length of the diagonals; and α_{s1} is the angle between the diagonals and the vertical as shown in **Figure 3.10**.

Figure 3.10 Evaluation of Shear Deformations

The horizontal deflection of column due to the shear distortion of this particular segment (δ_{s1}) is given by:

$$\delta_{s1} = \gamma_{s1} \times x_{s1} \quad (3.8)$$

where x_{s1} is the vertical distance of the region in estimating the average shear distortion as shown in **Figure 3.10**.

The total horizontal deflection due to shear distortions can thus be obtained through the summation of the individual segment deflection:

$$\delta_s = \sum \gamma_{si} \times x_{si} \quad (3.9)$$

where subscript ‘*i*’ represents the particular segment under consideration

3.7 Summary

This chapter describes an experimental program on ten 1/2-scale RC columns with light transverse reinforcement. The following provides a summary of the chapter:

1. The construction details of ten 1/2-scale RC columns with light transverse reinforcement were described together with details on the loading frame.
2. Areas affecting the performance of the test specimens were identified. Instrumentations were installed at various locations for measurement and thereafter determining the contribution of each factor on the performance of the test specimens.
3. The derivations of formula to estimate shear and flexure deformations based on the data obtained from the instrumentation were described in details.

CHAPTER 4

EXPERIMENTAL RESULTS

4.1 Introduction

This chapter presents the experimental results of the test specimens. Experimental results obtained include the measured hysteretic response (shear force versus lateral displacement), the observed cracking patterns, the strain readings from the reinforcing bars, the decomposition of horizontal displacements and the cumulative energy dissipations.

Figure 4.1 Definition of Performance Levels

The results of all test specimens will be presented together with performance levels as shown in **Figure 4.1**. Five performance levels at five significant parts of the test were identified. They are the drift ratio (DR) at which the cracking shear force (V_{cr}) is attained (PL1); drift ratio at which the theoretical yield force (V_y) is reached (PL2); drift ratio at which the maximum shear force (V_{max}) is attained (PL3); drift ratio at which the shear-resisting capacity drops more than 20% of V_{max} (PL4) and drift ratio at which the test specimen is unable to sustain the constant applied column axial load (PL5).

4.2 Test Results of Specimen SC-2.4-0.20

Specimen SC-2.4-0.20 had an aspect ratio (a/h) of 2.4. A column axial load of $0.20 f'_c A_g$ was applied to the specimen. The ratio of theoretical flexural strength to nominal shear strength (V_u/V_n) was 1.16. The concrete compressive strength of the specimen (f'_c) at the testing day was 22.6 MPa.

4.2.1 Hysteretic Response

Figure 4.2 Hysteretic Response of Specimen SC-2.4-0.20

Figure 4.2 shows the hysteretic response recorded from Specimen SC-2.4-0.20. The theoretical flexural strength (V_u) and nominal shear strength (V_n) of the specimen are also shown in **Figure 4.2**. The hysteretic loops show the degradation of stiffness and load-carrying capacity during repeated cycles due to the cracking of the concrete and yielding of the reinforcing bars. The pinching effect was observed in the hysteretic loops of the test specimen, which led to limited energy dissipation

as shown in **Figure 4.7**. The specimen did not reach its theoretical flexural strength up till the end of the test. In both the loading directions, the maximum shear force attained ($V_{\max}$) was approximately 218.9 kN, which was 94.7% of its theoretical flexural strength, corresponding to DRs of 1.26% in the positive direction and 1.45% in the negative direction. The specimen reached its theoretical yield force at DRs of 1.26% in the positive direction and 1.45% in the negative direction.

The shear-resisting capacity of the test specimen started to degrade at a DR of 1.84% in the negative loading direction. A significant loss of the shear-resisting capacity was observed at a DR of 1.98%. At this stage, the shear force was only 46.5% of $V_{\max}$. Continuous cycles caused additional damages and a loss of the shear-resisting capacity. The specimen was unable to sustain its constant applied axial load during the first cycle of a DR of 2.83%, which led to the test being stopped.

4.2.2 Cracking Patterns

Cracking patterns at each of the five performance levels described earlier are illustrated in **Figure 4.3**. When the specimen was loaded to a DR of 0.42% (PL1), the top and bottom of the specimen experienced an initiation of flexural cracks almost simultaneously. In the subsequent loading run corresponding to DRs of 0.80% and 1.01%, the specimen developed some diagonal shear cracks that appeared at both ends of the specimen. Limited new flexural cracks along the specimen were observed at this stage. In loading to a DR of 1.26% (PL2), additional diagonal shear cracks were developed; and the existing diagonal shear crack opened up and extended into the column at both ends. In loading to a DR of 1.44% (PL3), no significant changes in the crack patterns from the previous performance level (PL2) were observed. At a DR of 1.98% (PL4), a steep diagonal shear crack was formed at the middle of the specimen, which resulted in a loss of the shear-resisting capacity. Continuous cycles caused this diagonal shear crack opened up widely. In loading to a DR of 2.82%, the fracture of transverse reinforcements and buckling of longitudinal reinforcements were occurred along this diagonal shear crack. This led to the axial failure of the specimen.

at PL1
(DR=0.42%)

at PL2
(DR=1.26%)

at PL3
(DR=1.44%)

at PL4
(DR=1.98%)

at PL5
(DR=2.82%)

Figure 4.3 Observed Cracking Patterns at Different Performance Levels of Specimen SC-2.4-0.20

4.2.3 Strains in Longitudinal Reinforcing Bars

Figure 4.4 shows the measured strains along the longitudinal reinforcing bar of Specimen SC-2.4-0.20. It is to be noted that crushing and spalling of concrete at the column interfaces together with severe diagonal cracking at both ends of the specimen damaged a majority of strain gauges at near failure stages of the test. Therefore, the strain profile of the reinforcing bar is only shown up to a DR of 1.84%.

Figure 4.4 Local Strains in Longitudinal Reinforcing Bar of Specimen SC-2.4-0.20

The general strain profiles have showed to have good agreement with the bending moment pattern. It was observed that the measured strains along the longitudinal reinforcing bar varied considerably as drift ratios increased, apparently due to the growth of flexural cracks at both ends of the specimen.

With reference to this strain profile, tensile yielding of the longitudinal reinforcing bars was not observed during the tests. This indicated the dominance of shear in the failure behavior of the specimen. In loading to a DR of 1.23% (PL2), observed compressive strains at the column interfaces (L1 and L6) were slightly higher than tensile strains. At a DR of 1.84%, the compressive strain at L6 almost reached the yield strain.

4.2.4 Strains in Transverse Reinforcing Bars

Figure 4.5 shows the measured strains in the transverse reinforcing bars of Specimen SC-2.4-0.20. With reference to these strain profiles, yielding of the transverse steel bars was occurred at a DR of 1.44% (PL3). The largest tensile strain was recorded at 288 mm away from the column interface (T3). The strains in the transverse reinforcing bars increased drastically at a DR of 1.44% due to the growth and opening of diagonal shear cracks along the specimen.

Figure 4.5 Local Strains in Transverse Reinforcing Bars of Specimen SC-2.4-0.20

4.2.5 Displacement Decompositions

Figure 4.6 shows the contribution of deformation components expressed as percentages of the total lateral displacements at the peak displacements during each displacement cycle of Specimen SC-2.4-0.20. The definitions of each displacement component have been given in **Section 3.6**. It is to be noted that at a DR of 2.0%, crushing and spalling of concrete at both ends of the column together with severe diagonal cracking along the column induced false readings on a majority of the

measuring devices. Therefore, the displacement decompositions were only shown up to a DR of 2.0%.

Figure 4.6 Displacement Decompositions of Specimen SC-2.4-0.20

The results indicated that approximately 48 to 61% of the total lateral displacement was due to flexure. The shear displacement component was relatively small in the elastic range.

In loading to a DR of 1.44% (PL3), the shear displacement component increased significantly. At a DR of 1.98%, at which shear strength degradation became severe (PL4), the shear displacement component reached approximately 35% of the total lateral displacement.

4.2.6 Cumulative Energy Dissipation

The cumulative energy absorbed by Specimen SC-2.4-0.20 is shown in **Figure 4.7**. The total energy absorbed up to the point of axial failure was 34.9 kNm. At a DR of 1.26%, at which the maximum shear force occurred in the positive loading direction, the cumulative absorbed energy was only 25% of the total energy absorbed; and at a DR of 1.98% (PL4), it was 64.7% of the total energy absorbed.

Figure 4.7 Cumulative Energy Dissipation of Specimen SC-2.4-0.20

4.2.7 Summary of Specimen SC-2.4-0.20

In summary, this test showed that the shear failure was dominant in controlling the seismic behavior of the specimen. Based on the presented results, preliminary findings are as follows:

1. Specimen SC-2.4-0.20 reached the point of axial failure at a DR of 2.82%
2. The specimen did not reach its theoretical flexural strength up till the end of the test. In both the loading directions, the maximum shear force attained was approximately 218.9 kN, which was 85.4% of its theoretical flexural strength.
3. The total energy absorbed up to the point of axial failure was 34.9 kNm.
4. Approximately 48% to 61% of the total lateral displacement was due to flexure

4.3 Test Results of Specimen SC-2.4-0.50

Similar to Specimen SC-2.4-0.20, Specimen SC-2.4-0.50 had an aspect ratio of 2.4. A column axial load of $0.50 f'_c A_g$ was applied to the specimen. The ratio of theoretical flexural strength to nominal shear strength (V_u/V_n) was 1.00. The concrete compressive strength of the specimen (f'_c) at the testing day was 24.2 MPa.

4.3.1 Hysteretic Response

The hysteretic response together with the theoretical flexural strength and nominal shear strength of Specimen SC-2.4-0.50 are shown in **Figure 4.8**. Similar to Specimen SC-2.4-0.20, Specimen SC-2.4-0.50 showed a significant pinching behavior throughout the test.

Figure 4.8 Hysteretic Response of Specimen SC-2.4-0.50

Specimen SC-2.4-0.50 attained its yield force of 206.0 kN in both negative and positive loading directions at a DR of 0.82%. In the subsequent loading cycles, the specimen reached its maximum shear force of approximately 237.6 kN corresponding to a DR of 1.46% in both the loading directions. This maximum shear force was higher by 8.5% in comparison to Specimen SC-2.4-0.20 with a maximum shear force of 218.9 kN. The specimen reached its theoretical flexural strength at a DR of 1.46% in the positive loading direction.

In loading to the second cycle of a DR of 1.57%, the shear-resisting capacity of the specimen degraded significantly. At this stage, the shear forces were only 76.0% and 53.3% of its maximum shear force in the positive and negative loading directions, respectively. During the second cycle of a DR of 1.68%, the specimen was unable to sustain its applied column axial load, which led to the test being stopped.

4.3.2 Cracking Patterns

Figure 4.9 shows the cracking patterns at each of the five performance levels of Specimen SC-2.4-0.50. Together with the cracking patterns are the corresponding drift ratios for each of the performance levels.

At a DR of 0.25%, the specimen developed fine flexural cracks that were mostly concentrated along the bottom of the column. These flexural cracks propagated and spread along both ends of the column till a DR of 0.82%. In the subsequent loading cycles, fine shear cracks started developing at both ends of the column. No new flexural crack was observed at this stage.

In loading to a DR of 1.57%, a steep diagonal shear crack was formed at the middle of the specimen, which led to a loss of shear-resisting capacity in the specimen. In loading to a DR of 1.68%, the specimen failed to resist the applied column axial loads due to the buckling of longitudinal reinforcing bars at the bottom of the column.

at **PL1**
($DR=0.35\%$)

at **PL2**
($DR=0.82\%$)

at **PL3**
($DR=1.46\%$)

at **PL4**
($DR=1.68\%$)

at **PL5**
($DR=1.68\%$)

Figure 4.9 Observed Cracking Patterns at Different Performance Levels of Specimen SC-2.4-0.50

4.3.3 Strains in Longitudinal Reinforcing Bars

The measured strains along the longitudinal reinforcing bars in Specimen SC-2.4-0.50 are shown in **Figure 4.10**. Similar to Specimen SC-2.4-0.20, the general strain profiles of Specimen SC-2.4-0.50 have showed to have good agreement with the bending moment pattern.

Figure 4.10 Local Strains in Longitudinal Reinforcing Bars of Specimen SC-2.4-0.50

The largest recorded tensile strain of 2082 μ was observed at Location L6. It was slightly smaller as compared to Specimen SC-2.4-0.20 with the largest tensile strain of 2225 μ .

In loading to a DR of 1.01%, the compressive strain at Location L1 exceeded the compressive yield strain of -2545 μ . At the point of axial failure corresponding to a DR of 1.68%, the specimen attained its largest compressive strain of -8072 μ at Location L1. During the test, only compressive yielding was observed in the longitudinal reinforcing bars.

4.3.4 Strains in Transverse Reinforcing Bars

The measured strains in the transverse reinforcing bars of Specimen SC-2.4-0.50 are illustrated in **Figure 4.11**. It was observed that the measured strains varied considerably as drift ratios increased. The largest strain was detected at Location T2.

Figure 4.11 Local Strains in Transverse Reinforcing Bars of Specimen SC-2.4-0.50

The recorded strains in the transverse reinforcing bars were relatively small up to a DR of 1.46% (PL3). The largest recorded strain up to this stage was only 868 μ . In loading to a DR of 1.57% (PL4), the strains in the transverse reinforcing bars increased drastically due to the growth and opening of diagonal shear cracks along the specimen. Yielding of the transverse steel bars was only observed at the point of axial failure.

4.3.5 Displacement Decompositions

The contribution of deformation components expressed as percentages of the total lateral displacements at the peak displacements during each displacement cycle of

Specimen SC-2.4-0.50 is shown in **Figure 4.12**.

Approximately 62 to 69% of total lateral displacement was contributed by the flexural deformation component, whereas only up to 22% was accounted for by the shear deformation component. The shear deformation component initially grew gradually to approximately 10% of the total lateral displacement up to a DR of 1.46% (PL3). As the drift ratio was increased up to 1.68% (PL5), the corresponding shear deformation component grew to about 22% of the total displacement.

Figure 4.12 Displacement Decomposition of Specimen SC-2.4-0.50

4.3.6 Cumulative Energy Dissipation

The cumulative energy absorbed by Specimen SC-2.4-0.50 is shown in **Figure 4.13**. The total energy absorbed up to the point of axial failure was 26.3 kNm. This total energy was smaller by 24.6% in comparison to Specimen SC-2.4-0.20 with a total absorbed energy of 34.9 kNm. At a drift ratio of 1.46%, at which the maximum shear force occurred in both positive and negative loading directions, the absorbed energy was 66.7% of the total energy.

Figure 4.13 Cumulative Energy Dissipation of Specimen SC-2.4-0.50

4.3.7 Summary of Specimen SC-2.4-0.50

In summary, similar to the test results of Specimen SC-2.4-0.20, the shear failure was dominant in controlling the seismic behavior of the specimen. Based on the presented results, preliminary findings are as follows:

1. An increase of 8.5% in the maximum shear force was observed in Specimen SC-2.4-0.50 as compared to Specimen SC-2.4-0.20.
2. The specimen reached the point of axial failure at a DR of 1.68%, which was lower than that of Specimen SC-2.4-0.20.
3. The total energy absorbed up to the point of axial failure was 26.3 kNm, which was smaller by 24.6% in comparison to Specimen SC-2.4-0.20
4. Approximately 62 to 69% of total lateral displacement was contributed by the flexural deformation component

4.4 Test Results of Specimen SC-1.7-0.05

In SC-1.7 Series with an aspect ratio of 1.7, Specimen SC-1.7-0.05 had the smallest applied column axial load of $0.05 f'_c A_g$. The ratio of theoretical flexural strength to nominal shear strength (V_u/V_n) was 1.45. The concrete compressive strength of the specimen (f'_c) at the testing day was 29.8 MPa.

4.4.1 Hysteretic Response

The global behavior of the specimen can best be assessed by examining the hysteretic response. **Figure 4.14** shows the hysteretic response together with the flexural strength and nominal shear strength of Specimen SC-1.7-0.05.

Figure 4.14 Hysteretic Response of Specimen SC-1.7-0.05

A typical pinching behavior of shear-critical columns was observed throughout the test of Specimen SC-1.7-0.05. The specimen exceeded its yield force of 248.7 kN at

a DR of 0.98% in the negative loading direction. The maximum shear force attained was approximately 276.4 kN, which was 91.2% of its theoretical flexural strength, corresponding to a DR of 1.23% in the positive direction.

A sudden loss of shear-resisting capacity was observed at a DR of approximately 1.41% in both loading directions. At this stage, the attained shear forces were approximately 115.0 kN in the positive loading direction and 150.9 kN in the negative loading direction. In the subsequent loading cycles, the shear-resisting capacity reduced gradually. During the first cycle of a DR of 11.29%, the specimen was unable to sustain its applied column axial load, which led to the test being stopped.

4.4.2 Cracking Patterns

The general behavior of the specimen was based on the cracking patterns observed during the test. The cracking patterns at each of the performance levels together with the corresponding drift ratios of Specimen SC-1.7-0.05 are shown in **Figure 4.15**.

In loading to a DR of 0.41%, hairline flexural cracks were developed at the bottom of the column. These flexural cracks propagated till a DR of 0.66%. In the subsequent loading cycles, fine shear cracks occurred at both ends of the column. No new flexural crack was found at this stage.

In loading to a DR of 1.41% (PL4), steeper diagonal shear cracks were formed in the middle of the specimen, which led to a sudden loss of shear-resisting capacity. Damages associated with these diagonal shear cracks included the fracture of transverse reinforcing bars, buckling of longitudinal reinforcing bars and crushing of concrete along these wide cracks.

In loading to a DR of 11.29%, the specimen was unable to resist the applied column axial load, which led to the termination of the test.

at **PL1**
(DR=0.41%)

at **PL2**
(DR=0.98%)

at **PL3**
(DR=1.23%)

at **PL4**
(DR=1.41%)

at **PL5**
(DR=11.29%)

Figure 4.15 Observed Cracking Patterns at Different Performance Levels of Specimen SC-1.7-0.05

4.4.3 Strains in Longitudinal Reinforcing Bars

The recorded strains along the longitudinal reinforcing bars of Specimen SC-1.7-0.05 are shown in **Figure 4.16**. At a DR of 2.23%, a majority of strain gauges was damaged due to the crushing and spalling of concrete at the column interface together with severe diagonal cracking at both ends of the specimen. Therefore, the strain profiles of the reinforcing bars were only shown up to a DR of 2.23%.

Figure 4.16 Local Strains in Longitudinal Reinforcing Bars of Specimen SC-1.7-0.05

Strains in the longitudinal reinforcing bars of Specimen SC-1.7-0.05 and Specimen SC-2.4-0.20 were measured at similar locations as shown in **Figure 3.8**. The recorded strains along the longitudinal reinforcing bars in Specimen SC-1.7-0.05 increased gradually as drift ratios increased.

The largest tensile strain of 2450 μ was recorded at Location L1. It was slightly smaller than the yield strain of 2545 μ . The compressive yielding was observed at Location L1 during the loading to a DR of 1.29%.

4.4.4 Strains in Transverse Reinforcing Bars

Figure 4.17 shows the recorded strains in the transverse reinforcing bars of Specimen SC-1.7-0.05. Yielding was first occurred at a DR of 1.29%. The largest tensile strain was detected at T2 Location. The strains in the transverse reinforcing bars increased significantly at a DR of 1.00% due to the occurrence of diagonal shear cracks along the specimen.

Figure 4.17 Local Strains in Transverse Reinforcing Bars of Specimen SC-1.7-0.05

4.4.5 Displacement Decompositions

It is to be noted that at a DR of 2.0%, crushing and spalling of concrete at both ends of the column together with severe diagonal cracking along the column induced false readings on a majority of the measuring devices. Therefore, the displacement decompositions were only shown up to a DR of 2.0%. **Figure 4.18** shows the displacement decompositions of Specimen SC-1.7-0.05. Approximately 44 to 66% of total lateral displacement was contributed by the flexural deformation component, whereas 1.5 to 40.5% was accounted for by the shear deformation component.

Figure 4.18 Displacement Decomposition of Specimen SC-1.7-0.05

4.4.6 Cumulative Energy Dissipation

The cumulative energy absorbed by Specimen SC-1.7-0.05 is shown in **Figure 4.19**. The total energy absorbed by Specimen SC-1.7-0.05 was 35.1 kNm. At a DR of 1.23%, at which the maximum shear force occurred in the positive loading direction, the absorbed energy was 4.24 kNm, which was only 12.1% of the total energy.

Figure 4.19 Cumulative Energy Dissipation of Specimen SC-1.7-0.05

4.4.7 Summary of Specimen SC-1.7-0.05

Based on the presented results, preliminary findings are as follows:

1. A typical shear failure was observed throughout the test of Specimen SC-1.7-0.05.
2. Specimen SC-1.7-0.05 did not reach its theoretical flexural strength up till the end of the test. The maximum shear force attained was approximately 276.4 kN, which was 91.2% of its theoretical flexural strength.
3. Axial failure occurred at a DR of 11.29%, at which the lateral resistance of specimen was lost significantly.
4. The total energy absorbed by Specimen SC-1.7-0.05 was 35.1 kNm.
5. Approximately 44 to 66% of total lateral displacement was contributed by the flexural deformation component, whereas 1.5 to 40.5% was accounted for by the shear deformation component.

4.5 Test Results of Specimen SC-1.7-0.20

Specimen SC-1.7-0.20 had an aspect ratio of 1.7. A column axial load of $0.20 f'_c A_g$ was applied to the specimen. The ratio of theoretical flexural strength to nominal shear strength (V_u/V_n) was 1.26. The concrete compressive strength of the specimen (f'_c) at the testing day was 27.5 MPa.

4.5.1 Hysteretic Response

Figure 4.20 Hysteretic Response of Specimen SC-1.7-0.20

The hysteretic response of Specimen SC-1.7-0.20 together with its theoretical flexural strength and nominal shear strength are shown in **Figure 4.20**. The pinching effect was observed in the hysteretic loops of the test specimen, which led to limited energy dissipation as shown in **Figure 4.25**. In the negative direction, the maximum shear force attained was approximately 294.2 kN, which was 89.0% of its theoretical yield force or 83.3% of its theoretical flexural strength, corresponding

to a DR of 1.30%. In the positive direction, the specimen obtained a maximum shear force of 285.1 kN. The maximum shear force of Specimen SC-1.7-0.20 was higher by 6.4% in comparison to the one of Specimen SC-1.7-0.05.

A sudden loss of shear-resisting capacity was observed at a DR of 1.43%. Continuous loading caused additional damages and further loss of shear-resisting capacity. During the first cycle of a DR of 1.82% in the negative direction, the specimen was unable to sustain its applied column axial load, which led to the test being stopped. At this stage, the shear-resisting capacity was approximately zero.

4.5.2 Cracking Patterns

The general behavior of Specimen SC-1.7-0.20 was described through the cracking patterns together with the corresponding drift ratios observed during the test as shown in **Figure 4.21**. Specimen SC-1.7-0.20 does not have PL2 because the specimen did not reach its theoretical yield force up till the end of the test.

Hairline flexural cracks started to develop at the bottom of the column at a DR of 0.40%. The propagation of these flexural cracks was continued till a DR of 0.66%. In the subsequent loading cycles, a slight sign of shear inclination in the flexural cracks was observed at the bottom of the column. No new flexural crack was found at this stage. New diagonal shear cracks were formed at the bottom of the column during the loading run to a DR of 1.30% (PL3).

In loading to a DR of 1.43% (PL4), a steep diagonal shear crack was developed at the top of the column, whereas existing shear cracks at the bottom of the column opened up widely.

In loading to a DR of 1.82%, the fracture of transverse reinforcing bars, buckling of longitudinal reinforcing bars and crushing of concrete along the diagonal shear crack at the bottom of the column were observed, which led to the axial failure of the specimen and the termination of the test.

Figure 4.21 Observed Cracking Patterns at Different Performance Levels of Specimen SC-1.7-0.20

4.5.3 Strains in Longitudinal Reinforcing Bars

A majority of strain gauges was damaged due to the crushing and spalling of concrete at the column interface together with severe diagonal cracking at both ends of the specimen at a DR of 1.30%. Therefore, the recorded strains of reinforcing bars were only shown up to a DR of 1.30% as illustrated in **Figure 4.22** and **Figure 4.23**.

Figure 4.22 Local Strains in Longitudinal Reinforcing Bars of Specimen SC-1.7-0.20

The measured strains along the longitudinal reinforcing bar of Specimen SC-1.7-0.20 are shown in **Figure 4.22**. The recorded tensile strains were relatively small as compared to the compressive strains at the same locations throughout the test. The largest recorded tensile strain of 2010μ was observed at Location L1.

In loading to a DR of 1.30% (PL3), the compressive strain at Location L1 was -2661μ , which exceeded the compressive yield strain of -2545μ . The longitudinal reinforcing bars were only yielded compressively throughout the test.

4.5.4 Strains in Transverse Reinforcing Bars

The recorded strains in the transverse reinforcing bars of Specimen SC-1.7-0.20 are illustrated in **Figure 4.23**. The recorded strains in the transverse reinforcing bars were relatively small up to a DR of 0.85%, which agreed with the observed cracking patterns with limited shear cracks at this stage. The largest recorded strain up to this stage was only 292 μ . In the subsequent cycles, the strains in the transverse reinforcing bars were increased due to the formation of new shear cracks and the growth of existing shear cracks. Yielding of the transverse steel bars was observed at a DR of 1.30% (PL3).

Figure 4.23 Local Strains in Transverse Reinforcing Bars of Specimen SC-1.7-0.20

4.5.5 Displacement Decompositions

Figure 4.26 shows the contribution of deformation components expressed as percentages of the total lateral displacements at the peak displacements during each displacement cycle. It is to be noted that at a DR of 1.57%, crushing and spalling of concrete at both ends of the column together with severe diagonal cracking along the column induced false readings on a majority of the measuring devices.

Therefore, the displacement decompositions were only shown up to a DR of 1.57%.

The results indicate that approximately 48 to 66% of the total lateral displacement was due to flexure. The shear displacement component was about 8.0% in the elastic range. The shear displacement component increased significantly at a DR of 1.30%. At a DR of 1.57%, where severe shear strength degradation was observed, the shear displacement component reached approximately 35.5% of the total lateral displacement.

Figure 4.24 Displacement Decomposition of Specimen SC-1.7-0.20

4.5.6 Cumulative Energy Dissipation

Figure 4.25 shows the cumulative absorbed energy of Specimen SC-1.7-0.20. The total energy absorbed up to the point of axial failure was 13.5 kNm, which was smaller by 58.2% as compared to the one of Specimen SC-1.7-0.05. At a DR of 1.30%, at which the specimen attained the maximum shear force, the absorbed energy was 6.84 kNm (50.6% of the total energy).

Figure 4.25 Cumulative Energy Dissipation of Specimen SC-1.7-0.20

4.5.7 Summary of Specimen SC-1.7-0.20

Based on the presented results, preliminary findings are as follows:

1. An increase of 6.4% in the maximum shear force was observed in Specimen SC-1.7-0.20 as compared to Specimen SC-1.7-0.05.
2. Axial failure occurred at a DR of 1.82%, at which the lateral load resistance of the specimen was diminished. The drift ratio at axial failure of Specimen SC-1.7-0.20 was much lower than that of Specimen SC-1.7-0.05.
3. A higher maximum shear force and smaller drift ratio at axial failure was observed in Specimen SC-1.7-0.20 as compared to Specimen SC-2.4-0.20.
4. The total energy absorbed up to the point of axial failure was 13.5 kNm.
5. Approximately 48 to 66% of the total lateral displacement was due to flexure.

4.6 Test Results of Specimen SC-1.7-0.35

Specimen SC-1.7-0.35 had an aspect ratio of 1.7. A column axial load of $0.35 f'_c A_g$ was applied to the specimen. The ratio of theoretical flexural strength to nominal shear strength (V_u/V_n) was 1.17. The concrete compressive strength of the specimen (f'_c) at the testing day was 25.5 MPa.

4.6.1 Hysteretic Response

The hysteretic response together with the flexural strength and nominal shear strength of Specimen SC-1.7-0.35 are plotted in **Figure 4.26**. General trends of the graph were similar to that observed in Specimen SC-1.7-0.20. This was expected, as the only difference in these specimens was the column axial load.

Figure 4.26 Hysteretic Response of Specimen SC-1.7-0.35

Specimen SC-1.7-0.35 obtained a maximum shear force of 335.5 kN at a DR of 1.26%. The higher maximum shear force by approximately 14.0% achieved in Specimen SC-1.7-0.35 as compared to Specimen SC-1.7-0.20 was possibly due to the participation of the axial load in columns. Similar to Specimen SC-1.7-0.20, Specimen SC-1.7-0.35 did not reach its theoretical flexural strength throughout the test. Specimen SC-1.7-0.35 obtained its theoretical yield force at a DR of 1.26% in the negative loading direction.

The lateral loading resistance of the specimen was immediately lost after the shear strength of the specimen was attained. The axial failure occurred during the first cycle of a DR of 1.56% in the negative direction, which was lower than that of Specimen SC-1.7-0.20. The test was then terminated at this stage.

4.6.2 Cracking Patterns

Cracking patterns at each of the five performance levels of Specimen SC-1.7-0.35 are illustrated in **Figure 4.27**. The corresponding drift ratios for each of the performance levels are also shown in the same figure.

In loading to a DR of 0.33%, the flexural cracks initiated at the top and bottom of the specimen almost simultaneously. These flexural cracks propagated inward with a slight sign of shear inclination.

In the subsequent loading run, shear cracks propagating from the flexural cracks were continued developing. This was followed by more diagonal shear cracks with an angle of more than 45° forming at both ends of the column.

In loading to a DR of 1.44% (PL4), extensive shear cracking was developed at both ends of the column. Bond splitting cracks were formed at the middle of the column along the longitudinal reinforcing bars.

at **PL5**
($DR=1.56\%$)

Figure 4.27 Observed Cracking Patterns at Different Performance Levels of Specimen SC-1.7-0.35

4.6.3 Strains in Longitudinal Reinforcing Bars

A majority of strain gauges was damaged due to the crushing and spalling of concrete at the column interface together with severe diagonal cracking at both ends of the specimen at a DR of 1.30%. Therefore, the recorded strains of reinforcing bars were only shown up to a DR of 1.30%. The strain profiles along the longitudinal reinforcing bar of Specimen SC-1.7-0.35 are shown in **Figure 4.28**. Generally, a gradual increase in strains along the longitudinal reinforcing bar was observed as drift ratios increased.

Higher compressive strains than tensile strains were recorded at the column interfaces (L1 and L6). In loading to a DR of 1.26%, the strains at L1 and L6 Locations were yielded compressively. Tensile yielding of the longitudinal reinforcing bars was not observed throughout the test. This was expected, as a high column axial load of $0.35 f'_c A_g$ was applied to the specimen.

Figure 4.28 Local Strains in Longitudinal Reinforcing Bars of Specimen SC-1.7-0.35

4.6.4 Strains in Transverse Reinforcing Bars

The strain profiles of the transverse reinforcing bars of Specimen SC-1.7-0.35 are shown in **Figure 4.29**. Strains of transverse reinforcing bars at T2 and T5 Locations were exceeded the yield strain at a DR of 1.26%. A drastic increase in strain of transverse reinforcing bars was observed at a DR of 1.01% due to the occurrence of diagonal shear cracks along the specimen.

Figure 4.29 Local Strains in Transverse Reinforcements of Specimen SC-1.7-0.35

4.6.5 Displacement Decompositions

Figure 4.30 illustrate the contribution of displacement components expressed as percentages of the total lateral displacements at the peak displacements of Specimen SC-1.7-0.35. The definitions of each displacement component had been given in **Section 3.6**.

Similar to the previous two specimens in SC-1.7 Series, the major source of total lateral displacements was the flexure deformation. In Specimen SC-1.7-0.35, approximately 60 to 66% of the total lateral displacement was contributed by the flexural deformation component, whereas 2 to 22% was accounted for by the shear

deformation component.

Figure 4.30 Displacement Decomposition of Specimen SC-1.7-0.35

4.6.6 Cumulative Energy Dissipation

Figure 4.31 Cumulative Energy Dissipation of Specimen SC-1.7-0.35

Figure 4.31 shows the cumulative absorbed energy of Specimen SC-1.7-0.35. The total energy absorbed up to the point of axial failure was 9.09 kNm, which was 67.3% and 28.1% of Specimen SC-1.7-0.20 and SC-1.7-0.05's total energy, respectively. At a DR of 1.30%, at which the maximum shear force occurred, the absorbed energy was 5.51 kNm (60.6% of the total energy).

4.6.7 Summary of Specimen SC-1.7-0.35

Based on the presented results, preliminary findings are as follows:

1. An increase of 14.0% in the maximum shear force was observed in Specimen SC-1.7-0.35 as compared to Specimen SC-1.7-0.20.
2. Axial failure occurred at a DR of 1.56%, at which the lateral load resistance of the specimen was diminished. This drift ratio at axial failure was lower than that of Specimen SC-1.7-0.20.
3. The total energy absorbed up to the point of axial failure was 9.09 kNm.
4. Approximately 60 to 66% of the total lateral displacement was contributed by the flexural deformation component

4.7 Test Results of Specimen SC-1.7-0.50

Amongst all specimens in SC-1.7 Series, Specimen SC-1.7-0.50 had the highest applied column axial load of $0.50 f'_c A_g$. The ratio of theoretical flexural strength to nominal shear strength (V_u/V_n) was 1.02. The concrete compressive strength of the specimen (f'_c) at the testing day was 26.4 MPa.

4.7.1 Hysteretic Response

The hysteretic response together with the flexural strength and nominal shear strength of Specimen SC-1.7-0.50 are plotted in **Figure 4.32**. Specimen SC-1.7-0.50 showed a typical brittle shear failure and axial failure behaviors of reinforced concrete columns with light transverse reinforcement and a high column axial load.

Figure 4.32 Hysteretic Response of Specimen SC-1.7-0.50

A maximum shear force of 375.6 kN was obtained by Specimen SC-1.7-0.50 at a DR of 1.25%. The higher maximum shear force by approximately 12.0%, 27.7% and 35.9% achieved in Specimen SC-1.7-0.50 as compared to Specimen SC-1.7-0.35, SC-1.7-0.20 and SC-1.7-0.05, respectively was due to the effects of the column axial load.

As shown in **Figure 4.32**, the applied shear force in Specimen SC-1.7-0.50 exceeded its theoretical flexural strength at a DR of 1.25% in the negative loading direction. The lateral and axial loading resistance of the specimen was lost immediately after the specimen reached its maximum shear force. The ultimate recorded drift ratio obtained by the specimen was 1.42%. The test was then terminated at this stage.

4.7.2 Cracking Patterns

Cracking patterns of Specimen SC-1.7-0.50 are shown in **Figure 4.33**. The corresponding drift ratios for each of the performance levels are also shown in the same figure. At a DR of 0.26%, where the applied shear force exceeded the cracking force (PL1), there were no cracks formed. This could be due to the effects of a high axial load applied to the column of the specimen.

In loading to a DR of 0.67% the first flexural cracks formed at the bottom of column. In the subsequent loading run, these cracks propagated inward and started inclining. At a DR of 1.01% (PL2), extensive diagonal shear cracks with an angle of more than 45° formed at both top and bottom of the column. In loading to a DR of 1.25%, the cracking pattern of the specimen remained unchanged.

In loading to a DR of 1.42% (PL4, PL5), a wide shear cracks suddenly occurred, extending from top to bottom of the column. The fracture of transverse reinforcing bars and buckling of longitudinal reinforcing bars along the shear crack were occurred simultaneously with the formation of this crack, which led to the shear failure and axial failure of the specimen and the termination of the test.

at **PL5**
($DR=1.42\%$)

Figure 4.33 Observed Cracking Patterns at Different Performance Levels of Specimen SC-1.7-0.50

4.7.3 Strains in Longitudinal Reinforcing Bars

A majority of strain gauges was damaged due to the crushing and spalling of concrete at the column interface together with severe diagonal cracking at both ends of the specimen at a DR of 1.25%. Therefore, the recorded strains of reinforcing bars were only shown up to a DR of 1.25%.

The strain profiles along the longitudinal reinforcing bar of Specimen SC-1.7-0.50 are shown in **Figure 4.32**. Specimen SC-1.7-0.50 showed a similar trend in strains along the longitudinal reinforcing bars as compared to Specimen SC-1.7-0.35. Higher compressive strains than tensile strains were recorded at the column interface (L6).

In loading to a DR of 1.25% (PL3), the strain at L6 Locations was yielded compressively. Tensile yielding of the longitudinal reinforcing bars was not observed throughout the test. This was attributed to the effects of a high column axial force applied to the specimen.

Figure 4.34 Local Strains in Longitudinal Reinforcing Bars of Specimen SC-1.7-0.50

4.7.4 Strains in Transverse Reinforcing Bars

The measured strains in the transverse reinforcing bars of Specimen SC-1.7-0.50 are illustrated in **Figure 4.35**. It is to be noted the strain gauge at T3 Location is inoperative, possibly damaged during the casting process, which resulted in the missing data.

The largest strain was detected at Location T2. The recorded strains in the transverse reinforcing bars were relatively small up to a DR of 1.25% (PL3). The largest recorded strain up to this stage was only 961 μ . This complied with the cracking pattern at this stage (PL3), where little shear cracks were found.

Figure 4.35 Local Strains in Transverse Reinforcing Bars of Specimen SC-1.7-0.50

4.7.5 Displacement Decompositions

Figure 4.36 illustrate the contribution of displacement components expressed as percentages of the total lateral displacements at the peak displacements of Specimen SC-1.7-0.50. The major source of total lateral displacements was the flexure deformation. It is to be noted that at a DR of 1.25%, crushing and spalling of

concrete at both ends of the column together with severe diagonal cracking along the column induced false readings on a majority of the measuring devices. Therefore, the displacement decompositions were only shown up to a DR of 1.25%.

In Specimen SC-1.7-0.35, approximately 62 to 70% of the total lateral displacement was contributed by the flexural deformation component, whereas 1 to 7% was accounted for by the shear deformation component.

Figure 4.36 Displacement Decomposition of Specimen SC-1.7-0.50

4.7.6 Cumulative Energy Dissipation

Figure 4.37 shows the cumulative absorbed energy of Specimen SC-1.7-0.50. The total energy absorbed up to the point of axial failure was 4.16 kNm, which was 45.8% of Specimen SC-1.7-0.35's total energy. This could be attributed to a higher column axial load applied in Specimen SC-1.7-0.50 than in Specimen SC-1.7-0.35.

Figure 4.37 Cumulative Energy Dissipation of Specimen SC-1.7-0.50

4.7.7 Summary of Specimen SC-1.7-0.50

Based on the presented results, preliminary findings are as follows:

1. An increase of 12.0% in the maximum shear force was observed in Specimen SC-1.7-0.50 as compared to Specimen SC-1.7-0.35.
2. Shear and axial failure occurred at a DR of 1.42%, at which the lateral load resistance of the specimen was diminished. This drift ratio at axial failure was slightly lower than that of Specimen SC-1.7-0.35.
3. A higher maximum shear force and smaller drift ratio at axial failure was observed in Specimen SC-1.7-0.50 as compared to Specimen SC-2.4-0.50.
4. The total energy absorbed up to the point of axial failure was 4.16 kNm
5. Approximately 62 to 70% of the total lateral displacement was contributed by the flexural deformation component

4.8 Test Results of Specimen RC-1.7-0.05

The major difference between RC-1.7 Series and SC-1.7 Series was the cross sectional dimension as described in **Section 3.3.1**. RC-1.7 Series consisted of specimens with a cross sectional dimension of 250 mm x 490 mm, whereas the specimens in SC-1.7 Series had a cross sectional dimension of 350 mm x 350 mm.

In RC-1.7 Series, Specimen RC-1.7-0.05 had the smallest column axial load of $0.05 f'_c A_g$. The ratio of theoretical flexural strength to nominal shear strength (V_u / V_n) was 1.16. The concrete compressive strength of the specimen (f'_c) at the testing day was 32.5 MPa.

4.8.1 Hysteretic Response

The overall performance of the specimen can best be assessed by examining the hysteretic response. **Figure 4.38** illustrates the hysteretic response of Specimen RC-1.7-0.05. The theoretical flexural strength and nominal shear strength of Specimen RC-1.7-0.05 are also shown in the same figure.

It can be seen that with an increase in the applied lateral displacement, the shear force increased steadily. Up to a DR of 0.50%, no changes in the gradient of slope were observed. The specimen reached its theoretical yield force at a DR of 1.55% in both loading directions. A maximum shear force of 283.1 kN was obtained in the specimen at a DR of 1.98% in the positive loading direction. This was equivalent to 101.4% of its theoretical flexural strength.

In loading to a DR of 2.22%, a decrease in shear force was recorded. This decrease in shear force exceeded 20% of the maximum shear force at a DR of 2.49%. In the subsequent loading cycles, the specimen showed a gradual decrease in shear force with an increase in the applied lateral displacement.

In loading to a DR of 11.3%, the shear-resisting capacity of the specimen was only

20.5 kN, equivalent to 7.2% of its maximum shear force. Axial failure also occurred at this drift ratio. The test was then terminated at this stage.

Figure 4.38 Hysteretic Response of Specimen RC-1.7-0.05

4.8.2 Cracking Patterns

The general behavior of Specimen RC-1.7-0.05 was illustrated through the cracking patterns observed during the test. **Figure 4.39** shows the cracking patterns at each of the performance levels together with the corresponding drift ratios of the specimen.

When loading to a DR of 0.50% (PL1), fine flexural cracks were initiated at both ends of the column. Flexural cracks propagated horizontally in the columns with a slight sign of shear inclination observed at the top of the column. No shear cracks were observed at this stage.

at **PL1**
($DR=0.50\%$)

at **PL2**
($DR=1.55\%$)

at **PL3**
($DR=1.98\%$)

at **PL4**
($DR=2.49\%$)

at **PL5**
($DR=11.3\%$)

Figure 4.39 Observed Cracking Patterns at Different Performance Levels of Specimen RC-1.7-0.05

In loading to a DR of 1.55% (PL2), shear cracks propagating from the flexural cracks at both ends of the column were first observed. This was followed by more diagonal shear cracks at approximately 45°.

In loading to a DR of 1.98% (PL3), extensive shear cracks with an inclined angle of more than 45° were observed at both ends of the column. No new flexural cracks were formed at this stage. Bond splitting cracks were developed at the middle of the column along the centered longitudinal reinforcing bar.

In loading to a DR of 2.49% (PL4), the bond splitting cracks along the centered longitudinal reinforcing bar were appeared visibly. A new bond splitting cracks was formed along the side longitudinal reinforcing bar. No new shear and flexural cracks were observed at this stage.

In loading to a DR of 11.3%, spalling of concrete cover along the bond splitting crack was observed. Crushing of concrete together with fracturing of transverse reinforcing bars along the diagonal shear cracks was recorded. At this stage, the specimen had reached its axial failure.

4.8.3 Strains in Longitudinal Reinforcing Bars

The measured strains in the longitudinal reinforcing bar of Specimen RC-1.7-0.05 are illustrated in **Figure 4.40**. It is to be noted the strain gauges at L3 and L5 Locations are inoperative, possibly damaged during the casting process, which resulted in the missing data. A majority of strain gauges was damaged due to the crushing and spalling of concrete at the column interface together with severe diagonal cracking at both ends of the specimen at a DR of 2.49%. Therefore, the recorded strains of reinforcing bars were only shown up to a DR of 2.49%.

It can be seen that as drift ratios increased, strains in the longitudinal reinforcing bars increased gradually. Tensile and compressive yielding was observed at L1 and L6 Locations when the drift ratio exceeded 1.98% (PL3). In loading to a DR of

2.49% (PL4), the tensile strain in the longitudinal reinforcing bar at L2 Location almost reached the yield strain of 2545 μ .

Figure 4.40 Local Strains in Longitudinal Reinforcing Bars of Specimen RC-1.7-0.05

4.8.4 Strains in Transverse Reinforcing Bars

The measured strains in the transverse reinforcing bars of Specimen RC-1.7-0.05 are illustrated in **Figure 4.41**. The locations of the strain gauges are also plotted in **Figure 4.41**.

As shown in **Figure 4.41**, the strains at T1 and T6 Locations were very small throughout the test. It complied with the cracking patterns as shown in **Figure 4.39**, where little shear cracks were found at these locations. The strains in transverse reinforcing bars were relatively small up to a DR of 1.03%. The largest strain in transverse reinforcing bars at this stage was 634 μ .

In the subsequent drift ratio, a drastic increase in strains was observed. It is to be noted that at this stage extensive shear cracks occurred at both ends of the column.

The sudden increase in strains was due to the occurrence of these shear cracks. In loading to a DR of 1.98% (PL3), yielding was observed at T4 and T5 Locations. The strain at T2 Location was almost reached the yield strain at this stage. At a DR of 2.49% (PL4), the strain at T2 Location exceeded the yield strain.

Figure 4.41 Local Strains in Transverse Reinforcing Bars of Specimen RC-1.7-0.05

4.8.5 Displacement Decompositions

Figure 4.42 shows the contribution of deformation components expressed as percentages of the total lateral displacements at the peak displacements during each displacement cycle of Specimen RC-1.7-0.05. The definitions of each displacement component had been given in **Section 3.6**.

It is to be noted that crashing and spalling of concrete at both ends of the column together with severe diagonal cracking along the column induced false readings on a majority of the measuring devices at a DR of 2.49%. Therefore, the displacement decompositions were only shown up to a DR of 2.49%.

The results indicated that approximately 40 to 55% of the total lateral displacement was due to flexure. The shear displacement component was relatively small up till a

DR of 1.0%. In loading to a DR of 1.98% (PL3), the shear displacement component increased significantly. At a DR of 2.49%, at which shear strength degradation became severe (PL4), the shear displacement component reached approximately 29% of the total lateral displacement.

Figure 4.42 Displacement Decomposition of Specimen RC-1.7-0.05

4.8.6 Cumulative Energy Dissipation

Figure 4.43 Cumulative Energy Dissipation of Specimen RC-1.7-0.05

The cumulative energy absorbed by Specimen RC-1.7-0.05 is shown in **Figure 4.43**. The total energy absorbed of Specimen RC-1.7-0.05 was 77.1 kNm, which was higher than that of Specimen SC-1.7-0.05. At a DR of 1.98%, at which the maximum shear force occurred in the positive loading direction, the cumulative absorbed energy was 21.7 kNm, equivalent to 28.1% of the total energy absorbed; and at a DR of 1.98% (PL4), it was 40.8% of the total energy absorbed.

4.8.7 Summary of Specimen RC-1.7-0.05

Based on the presented results, preliminary findings are as follows:

1. The failure of Specimen RC-1.7-0.05 was controlled by the shear cracking at both ends of the column and bond splitting along the column.
2. The axial failure of Specimen RC-1.7-0.05 occurred at a DR of 11.3%.
3. A maximum shear force of 283.1 kN was obtained in the specimen at a DR of 1.98% in the positive loading direction. This was equivalent to 101.4% of its theoretical flexural strength.
4. The total energy absorbed of Specimen RC-1.7-0.05 was 77.1 kNm.
5. Approximately 40 to 55% of the total lateral displacement was due to flexure.

4.9 Test Results of Specimen RC-1.7-0.20

Similar to Specimen RC-1.7-0.05, Specimen RC-1.7-0.20 had a cross sectional dimension of 250 mm x 490 mm. A moderate column axial load of $0.20 f'_c A_g$ was applied to the specimen. The ratio of theoretical flexural strength to nominal shear strength (V_u/V_n) was 1.05. The concrete compressive strength of the specimen (f'_c) at the testing day was 24.5 MPa.

4.9.1 Hysteretic Response

Figure 4.44 illustrates the hysteretic response of Specimen RC-1.7-0.20. The theoretical flexural strength and nominal shear strength of Specimen RC-1.7-0.05 are also shown in the same figure.

Figure 4.44 Hysteretic Response of Specimen RC-1.7-0.20

The hysteretic loops of Specimen RC-1.7-0.20 showed the degradation of stiffness and load-carrying capacity during repeated cycles. The pinching effect was observed in the hysteretic loops of the test specimen, which led to limited energy dissipation as shown in **Figure 4.49**.

The maximum shear force attained was approximately 305.5 kN, which was 97.4% of its theoretical flexural strength, corresponding to DR of 1.57% in the negative direction. The higher maximum shear force by approximately 7.9% achieved in Specimen RC-1.7-0.20 as compared to Specimen RC-1.7-0.05 was due to the effects of the axial loads in columns.

In loading to a DR of 2.30%, the shear-resisting capacity of the specimen degraded more than 20% of the maximum shear force. Continuous cycles caused additional damages and a loss of the shear-resisting capacity. At a DR of 2.87%, the shear-resisting capacity of the specimen was 50% of its maximum shear force. During the second cycle of a DR of 2.87%, the shear-resisting capacity of the specimen was only 31.5% of its maximum shear force. At this stage, the axial failure was occurred, which led to the termination of the test.

4.9.2 Cracking Patterns

The cracking patterns at each of the performance levels together with the corresponding drift ratios of Specimen RC-1.7-0.20 are shown in **Figure 4.45**. The general trends of the cracking patterns of Specimen RC-1.7-0.20 were similar to that observed in Specimen RC-1.7.0.05.

Hairline flexural cracks were initiated at both ends of the column at a DR of 0.34% (PL1). Flexural cracks propagated horizontally in the columns with no sign of shear inclination observed at both ends of the column. No shear cracks were observed at this stage. The flexural cracks started inclining inward at a DR of 0.67%. In loading to a DR of 1.01%, extensive shear cracking was observed at both ends of the column.

at **PL1**
($DR=0.34\%$)

at **PL2**
($DR=1.44\%$)

at **PL3**
($DR=1.56\%$)

at **PL4**
($DR=2.30\%$)

at **PL5**
($DR=2.87\%$)

Figure 4.45 Observed Cracking Patterns at Different Performance Levels of Specimen RC-1.7-0.20

In loading to a DR of 1.44% (PL2), shear cracks with a steep angle of were initiated extensively at both ends of the column. These cracks expanded to a distance of 650 mm away from the column interfaces. Little new flexural cracks were found at this stage.

In loading to a DR of 2.30% (PL4), the bond splitting cracks along the centered longitudinal reinforcing bar were observed at the middle of the column along the centered longitudinal reinforcing bar. This crack was the extension of the existing shear crack at the top of the column. New diagonal cracks with a steep angle were also found at stage.

In loading to a DR of 2.87% (PL5), significant spalling of concrete cover along both sides of the bottom of column together with crushing of concrete at the bottom was observed. Fracturing of transverse reinforcing bars at the bottom of column was also seen at this stage of loading.

4.9.3 Strains in Longitudinal Reinforcing Bars

The measured strains in the longitudinal reinforcing bars of Specimen RC-1.7-0.20 are illustrated in **Figure 4.46**. The locations of the strain gauges are also plotted in **Figure 4.46**. A majority of strain gauges was damaged due to the crushing and spalling of concrete at the column interface together with severe diagonal cracking at both ends of the specimen at PL4. Therefore, the recorded strains of reinforcing bars were only shown up to PL4.

It was observed that the measured strains along the longitudinal reinforcing bars varied considerably as drift ratios increased, apparently due to the growth of flexural cracks at both ends of the specimen. With reference to this strain profile, tensile yielding of the longitudinal reinforcing bars was observed at a DR of 2.30%.

In loading to a DR of 1.56%, the compressive strains at L1 and L6 Locations were almost exceeded the yield strain of -2545μ . In the subsequent loading cycles,

compressive yielding was observed at these locations. The largest recorded compressive strain in the transverse reinforcing bars up to a DR of 2.46% was -3658μ .

Figure 4.46 Local Strains in Longitudinal Reinforcing Bars of Specimen RC-1.7-0.20

4.9.4 Strains in Transverse Reinforcing Bars

The measured strains in the transverse reinforcing bars of Specimen RC-1.7-0.20 are illustrated in **Figure 4.47**. The locations of the strain gauges are also plotted in **Figure 4.47**.

The recorded strains in the transverse reinforcing bars were relatively small up to a DR of 0.82%, which agreed with the observed cracking patterns with limited shear cracks at this stage. The largest strain at this stage was 504μ . In the subsequent cycles, the formation of new shear cracks and growth of existing shear cracks increased the strains in the transverse reinforcing bars.

In loading to a DR of 2.02%, yielding of the transverse steel bars was first observed at T3 Location. At a DR of 2.45%, strain at T2 Location also exceeded the yield strain.

Figure 4.47 Local Strains in Transverse Reinforcing Bars of Specimen RC-1.7-0.20

4.9.5 Displacement Decompositions

Figure 4.48 Displacement Decomposition of Specimen RC-1.7-0.20

It is to be noted that crashing and spalling of concrete at both ends of the column together with severe diagonal cracking along the column induced false readings on a majority of the measuring devices at a DR of 2.5%. Therefore, the displacement decompositions were only shown up to a DR of 2.5%. The contribution of deformation components expressed as percentages of the total lateral displacements at the peak displacements during each displacement cycle is shown in **Figure 4.48**. Approximately 40 to 53% of the total lateral displacement was contributed by the flexural deformation component, whereas 2 to 38% was accounted for by the shear deformation component. The shear deformation component initially was relatively small. As drift ratios increased, the shear deformation increased drastically. It reached 38% of the total displacement at a DR of 2.5%.

4.9.6 Cumulative Energy Dissipation

The cumulative energy absorbed by Specimen RC-1.7-0.20 is shown in **Figure 4.48**. The total energy absorbed up to the axial failure stage was 44.3 kNm, which was smaller than that of Specimen RC-1.7-0.05. At a DR of 1.53%, at which the maximum shear force occurred in the negative loading direction, the cumulative absorbed energy was 20.0 kNm, equivalent to 45.2% of the total energy absorbed.

Figure 4.49 Cumulative Energy Dissipation of Specimen RC-1.7-0.20

4.9.7 Summary of Specimen RC-1.7-0.20

Based on the presented results, preliminary findings are as follows:

1. The failure of Specimen RC-1.7-0.20 was controlled by the shear cracking at both ends of the column
2. An increase of 7.9% in the maximum shear force was observed in Specimen RC-1.7-0.20 as compared to Specimen RC-1.7-0.05.
3. Axial failure occurred at a DR of 2.87%, which was much lower than that of Specimen RC-1.7-0.05.
4. The total energy absorbed up to the axial failure stage was 44.3 kNm, which was smaller than that of Specimen RC-1.7-0.05.
5. Approximately 40 to 53% of the total lateral displacement was contributed by the flexural deformation component, whereas 2 to 38% was accounted for by the shear deformation component.

4.10 Test Results of Specimen RC-1.7-0.35

A column axial load of $0.35 f'_c A_g$ was applied to Specimen RC-1.7-0.35. The ratio of theoretical flexural strength to nominal shear strength (V_u/V_n) was 1.00. The concrete compressive strength of the specimen (f'_c) at the testing day was 27.1 MPa.

4.10.1 Hysteretic Response

The shear force versus lateral displacement response of Specimen RC-1.7-0.35 is plotted in **Figure 4.50**. In general, Specimen RC-1.7-0.35 showed a typical brittle shear failure response.

Figure 4.50 Hysteretic Response of Specimen RC-1.7-0.35

A gradual increase in the shear force of the specimen with an increase in the applied lateral displacement was observed up to a DR of 1.01% (PL2). Slight decrease in the gradient of the backbone curve was observed just after a DR of 1.01%. A

maximum shear force of 345.7 kN was recorded at a DR of 1.45% (PL3) in the negative loading direction. The ratio of the maximum shear force to theoretical flexural strength was 0.98. A higher maximum shear force of approximately 13.2% was achieved in Specimen RC-1.7-0.35 as compared to Specimen RC-1.7-0.20.

Specimen RC-1.7-0.35 suddenly lost its shear strength just after a DR of 1.45% (PL3). In loading to a DR of 1.65%, the shear strength of Specimen RC-1.7-0.35 was only 210.1 kN, corresponding to 60.8% of its maximum shear force. In the subsequent loading run, the shear strength of the specimen reduced gradually. At point of axial failure corresponding to a DR of 2.02%, the shear strength of the test specimen was around 139.7 kN.

4.10.2 Cracking Patterns

The cracking patterns at five performance levels together with the corresponding drift ratios of Specimen RC-1.7-0.35 are shown in **Figure 4.51**. In general, the cracking patterns of Specimen RC-1.7-0.35 showed distinctively different trends to that observed in Specimens RC-1.7.0.05 and RC-1.7-0.20.

In loading to a DR of 0.26%, hairline flexural cracks were developed at both ends of the column. These flexural cracks propagated with a sign of inclination till a DR of 1.01%. In loading to a DR of 1.45% (PL3), steep diagonal shear cracks with an angle of more than 45° were found at the top of the column. No new flexural cracks were found at this stage.

In loading to a DR of 1.65%, severe shear cracking was developed at the top of the column. A shear crack with a wide opening at the top of the column was observed. The sudden decrease in shear-resisting capacity was deemed to the occurrence of this wide shear crack. In the subsequent drift ratio, all damages were accumulated along this wide shear crack. These damages included fracturing of transverse reinforcing bars, buckling of longitudinal reinforcing bars and crushing of concrete along these wide cracks. In loading to a DR of 2.02%, the specimen was unable to resist the applied axial load, which led to the termination of the test.

Figure 4.51 Observed Cracking Patterns at Different Performance Levels of Specimen RC-1.7-0.35

4.10.3 Strains in Longitudinal Reinforcing Bars

The measured strains along the longitudinal reinforcing bars in the specimen are shown in **Figure 4.52**. In general, strain profiles of Specimen RC-1.7-0.35 have showed to have good agreement with the bending moment pattern. The largest recorded tensile strain of 2700μ was observed at Location L6. Both tensile and compressive yielding were observed in longitudinal reinforcing bars.

Figure 4.52 Local Strains in Longitudinal Reinforcing Bars of Specimen RC-1.7-0.35

4.10.4 Strains in Transverse Reinforcing Bars

Figure 4.53 shows the recorded strains in the transverse reinforcing bars of Specimen RC-1.7-0.35. Yielding was first occurred at a DR of 1.58%. The largest tensile strain was detected at T4 Location. The strains in the transverse reinforcing bars increased significantly at a DR of 1.58% due to the occurrence of diagonal shear cracks along the specimen.

Figure 4.53 Local Strains in Transverse Reinforcing Bars of Specimen RC-1.7-0.35

4.10.5 Displacement Decompositions

Figure 4.54 shows the displacement decompositions of Specimen RC-1.7-0.35. Approximately 43 to 51% of the total lateral displacement was contributed by the flexural deformation component, whereas 2.5 to 32.5% was accounted for by the shear deformation component.

Figure 4.54 Displacement Decomposition of Specimen RC-1.7-0.35

4.10.6 Cumulative Energy Dissipation

The cumulative energy absorbed by Specimen RC-1.7-0.35 is shown in **Figure 4.55**. The total energy absorbed up to the axial failure stage was 26.5 kNm, which was smaller than that of Specimen RC-1.7-0.20.

Figure 4.55 Cumulative Energy Dissipation of Specimen RC-1.7-0.35

4.10.7 Summary of Specimen RC-1.7-0.35

Based on the presented results, preliminary findings are as follows:

1. An increase of 13.2% in the maximum shear force was observed in Specimen RC-1.7-0.35 as compared to Specimen RC-1.7-0.20.
2. Axial failure occurred at a DR of 2.02%, which was lower than that of Specimen RC-1.7-0.20.
3. The total energy absorbed up to the axial failure stage was 26.5 kNm, which was smaller than that of Specimen RC-1.7-0.20.
4. Approximately 43 to 51% of the total lateral displacement was contributed by the flexural deformation component.

4.11 Test Results of Specimen RC-1.7-0.50

Amongst all specimens in RC-1.7 Series, Specimen RC-1.7-0.50 had the highest applied column axial load of $0.50 f'_c A_g$. The ratio of theoretical flexural strength to nominal shear strength (V_u/V_n) was 0.85. The concrete compressive strength of the specimen (f'_c) at the testing day was 26.8 MPa.

4.11.1 Hysteretic Response

The shear force versus lateral displacement response of Specimen RC-1.7-0.50 is plotted in **Figure 4.56**. General trends of the graph were similar to that observed in the previous specimen (RC-1.7-0.35).

Figure 4.56 Hysteretic Response of Specimen RC-1.7-0.50

Similar to Specimen RC-1.7-0.35, a gradual increase in the shear force of the specimen with an increase in the applied lateral displacement was observed up to

PL2 (a DR of 0.79%). After that, a decrease in the gradient of the backbone curve was observed. A maximum shear force of 355.2 kN was recorded at a DR of 1.44% (PL3) in the negative loading direction. It was slightly higher than that recorded in Specimen SC-1.7-0.35.

Both Specimens RC-1.7-0.35 and RC-1.7-0.50 depicted a brittle shear failure behavior. The shear-resisting capacity of Specimen RC-1.7-0.50 was suddenly reduced at a DR of 1.67% just after the maximum shear force was reached. This was followed by a gradual decrease in the shear strength of the test specimen. At a DR of 1.80%, the specimen reached its axial failure.

4.11.2 Cracking Patterns

The cracking patterns at five performance levels together with the corresponding drift ratios of Specimen RC-1.7-0.50 are shown in **Figure 4.57**. Generally, similar trends of the cracking patterns were observed in Specimen RC-1.7-0.50 as compared to that observed in Specimens RC-1.7.0.05 and RC-1.7.0.20.

Hairline flexural cracks were developed at both ends of the column at a DR of 0.20% (PL1). Flexural cracks propagated horizontally in the columns with no sign of shear inclination observed at both ends of the column till a DR of 0.79% (PL2). In loading to a DR of 1.44% (PL3), severe shear cracking was initiated at both ends of the column. In loading to a DR of 1.67% (PL4), the bond splitting crack along the centered longitudinal reinforcing bar were observed in the middle of the column along the centered longitudinal reinforcing bar. This crack was the extension of the existing shear crack at the top of the column. New diagonal cracks with a steep angle were also found at stage.

In loading to a DR of 1.80% (PL5), significant spalling of concrete cover along both sides of the bottom of column together with crushing of concrete at the bottom was observed. Fracturing of transverse reinforcing bars together with buckling of longitudinal reinforcing bars at the bottom of column was also seen at this stage.

at **PL1**
($DR=0.20\%$)

at **PL2**
($DR=0.79\%$)

at **PL3**
($DR=1.44\%$)

at **PL4**
($DR=1.67\%$)

at **PL5**
($DR=1.80\%$)

Figure 4.57 Observed Cracking Patterns at Different Performance Levels of Specimen RC-1.7-0.50

4.11.3 Strains in Longitudinal Reinforcing Bars

The measured strains in the longitudinal reinforcing bars of Specimen RC-1.7-0.50 are illustrated in **Figure 4.58**. With reference to this strain profile, tensile yielding of the longitudinal reinforcing bars was observed up till a DR of 1.67%; whereas compressive yielding was occurred at a DR of 1.03%.

Figure 4.58 Local Strains in Longitudinal Reinforcing Bars of Specimen RC-1.7-0.50

4.11.4 Strains in Transverse Reinforcing Bars

Figure 4.59 shows the recorded strains in the transverse reinforcing bars of Specimen RC-1.7-0.50. Yielding was first occurred at a DR of 1.57%. The strains in the transverse reinforcing bars increased significantly at a DR of 1.57% due to the occurrence of diagonal shear cracks along the specimen.

Figure 4.59 Local Strains in Transverse Reinforcing Bars of Specimen RC-1.7-0.50

4.11.5 Displacement Decompositions

Figure 4.60 shows the displacement decompositions of Specimen RC-1.7-0.50. Approximately 48 to 60% of the total lateral displacement was contributed by the flexural deformation component, whereas 2 to 39% was accounted for by the shear deformation component.

Figure 4.60 Displacement Decomposition of Specimen RC-1.7-0.50

4.11.6 Cumulative Energy Dissipation

The cumulative energy absorbed by Specimen RC-1.7-0.50 is shown in **Figure 4.61**. The total energy absorbed up to the axial failure stage was 23.6 kNm, which was slightly smaller than that of Specimen RC-1.7-0.35.

Figure 4.61 Cumulative Energy Dissipation of Specimen RC-1.7-0.50

4.11.7 Summary of Specimen RC-1.7-0.50

Based on the presented results, preliminary findings are as follows:

1. A slight increase in the maximum shear force was observed in Specimen RC-1.7-0.50 as compared to Specimen RC-1.7-0.35.
2. Axial failure occurred at a DR of 1.80%, which was slightly lower than that of Specimen RC-1.7-0.35.
3. The total energy absorbed up to the axial failure stage was 23.6 kNm, which was slightly smaller than that of Specimen RC-1.7-0.35.

4. Approximately 48 to 60% of the total lateral displacement was contributed by the flexural deformation component.

4.12 Summary

This chapter presents the results of the test specimens from three different series. The hysteretic response, cracking patterns, strains in reinforcing bars, displacement decompositions and cumulative energy dissipation of each test specimen were discussed in details.

Further comparisons between the specimens are necessary to determine the influences of the various parameters, namely the column axial load, cross sectional shape and aspect ratio on the seismic behavior of the test specimens.

CHAPTER 5

DISCUSSION AND COMPARISON OF EXPERIMENTAL RESULTS

5.1 Introduction

The test results of the ten reinforced concrete (RC) columns with light transverse reinforcement were reported individually in the previous chapter. In this chapter, further discussion and investigation will be carried out to establish deeper understanding of the seismic behavior of the RC columns with light transverse reinforcement subjected to seismic loadings. Selected results from all test specimens will be compared in this chapter to determine the effects of column axial load, aspect ratio and cross sectional shape on the seismic performance of the test columns. The backbone curves obtained from the experimental results of all test specimens are also compared with FEMA 356 [F1] and ASCE [E8]'s models.

5.2 Comparison of Cracking Patterns

The cracking patterns of all test specimens had been illustrated in the previous chapter (**Chapter 4**). In general, the observed cracking patterns can be divided into five stages: shear cracking (PL1), yielding (PL2), maximum response (PL3), shear failure (PL4) and axial failure (PL5). The cracking patterns of the test specimens at each of the five stages will be compared in this part of the chapter.

In loading to PL1, all test specimen developed fine flexural cracks concentrated at both ends of the columns. The lower the applied axial load was, the more flexural cracks were observed in the columns. A slight sign of inclination in these flexural cracks were seen in most of the test specimen at this stage. These were no distinctive differences in the cracking patterns of the test specimens at this stage.

It is to be noted that Specimen SC-1.7-0.20 did not reach its theoretical yield force till the end of their tests. Therefore, the comparison of the cracking patterns at PL2 is applicable to all specimens except this specimen. In loading to PL2, while the specimens with a low axial load developed severe shear cracking at both ends of the columns, the specimens with a high axial load only showed a slight sign of shear inclination in the flexural cracks.

In loading to PL3, severe shear cracking was occurred at both ends of the column in all the test specimens. The lower the column axial load was, the longer these shear cracks were extended at both ends of the column.

(a) Shear Cracking

(b) Shear and Bond-Splitting Cracking

Figure 5.1 Modes of Shear Failure in Test Specimens

In loading to PL4, while the occurrence of a steep shear crack and the opening of the existing shear cracks resulted in a decrease in the shear-resisting capacity of SC-

1.7 and SC-2.4 Series specimens as shown in **Figure 5.1(a)**, the shear failure in all RC-1.7 Series specimens was controlled by a combination of shear and bond-splitting cracking as illustrated in **Figure 5.1 (b)**.

As shown in **Figure 5.2**, there are two axial failure modes observed in the test specimens. In the first mode of axial failure, the steep shear crack developed on the column from the previous stages became wider. This led to sliding between the crack surfaces as well as buckling of longitudinal reinforcing bars and fracturing of transverse reinforcing bars along this shear crack. In the second mode of axial failure, crushing of concrete as well as the buckling of longitudinal reinforcing bars and fracturing of transverse developed across a damaged zone. This type of axial failure was observed in most of RC-1.7 Series specimens, whereas most of SC-1.7 and SC-2.4 Series specimens depicted the first mode of axial failure.

(a) Mode 1

(b) Mode 2

Figure 5.2 Modes of Axial Failure in Test Specimens

5.3 Comparison of Backbone Curves

Figures 5.3, 5.4 and 5.5 show the comparison of the backbone curves of the test specimens in SC-2.4, SC-1.7, and RC-1.7 Series; respectively. Comparisons were made based on the general profile of the curves, initial stiffness, shear strength, and drift ratio at axial failure of the test specimens. The initial stiffness, shear strength and drift ratio at axial failure of all test specimens are also summarized in Table 5.1.

5.3.1 General Profile of the Backbone Curves

As illustrated in Figures 5.3, 5.4 and 5.5, the shear failure in all test specimens occurred at a DR of less than 2.0%. Typical brittle-failure backbone curves were observed in all test specimens. The backbone curves of all test specimens were generally similar.

Figure 5.3 Backbone Curves of SC-2.4 Series Specimens

Figure 5.4 Backbone Curves of SC-1.7 Series Specimens

Figure 5.5 Backbone Curves of RC-1.7 Series Specimens

However, some differences were detected in the trends of these curves. For the specimens in both SC-2.4 and RC-1.7 Series, before reaching their maximum shear forces, several distinct changes in the gradient of slope were observed. However, no distinct change in the gradient of slope was observed in the specimens of SC-1.7 Series. In addition, it was observed that the steeper negative gradient was recorded in the specimens of SC-1.7 Series as compared to the specimens of SC-2.4 and RC-1.7 Series with the same column axial load.

As shown in **Figures 5.3, 5.4 and 5.5**, the positive and negative gradients of the curves were significantly different with the change in column axial load. In all series, the higher the column axial load was, the steeper the negative and positive gradients were observed.

5.3.2 Initial Stiffness

The initial stiffness was calculated based on a point obtained from the measured force-displacement envelope with a shear force that is equal to the theoretical yield force. This is defined as either the first yield that occurs within the longitudinal reinforcement or when the maximum compressive strain of the concrete attains a value of 0.002 at any critical section of the column. This definition could not be used for columns whose shear strength does not substantially exceed its theoretical yield force. For such columns, defined as those whose maximum measured shear force was less than 107% of the theoretical yield force, the effective stiffness was defined based on a point on its measured force-displacement envelope with a shear force that equates to 80% of the obtained maximum shear force.

The relationships between initial stiffness and the column axial load ratio of all test specimens are plotted in **Figure 5.6**. The initial stiffness of SC-1.7 Series specimens were enhanced by around 9.8%, 17.6%, and 40.4% as the column axial load was increased from 0.05 to 0.20, 0.35, and $0.50 f'_c A_g$, respectively. An analogous trend was observed in the specimens of RC-1.7 Series, whose initial stiffness experienced an enhancement of around 33.9%, 64.3% and 86.1% with an increase in the column

axial load from 0.05 to 0.20, 0.35 and 0.50 $f'_c A_g$, respectively. As compared to Specimen SC-2.4-0.20, Specimen SC-2.4-0.50 experienced a 20.2% increase in its initial stiffness. This clearly indicates that the column axial load was beneficial to the initial stiffness of the test specimens.

Figure 5.6 Comparison of Initial Stiffness between Test Specimens

The initial stiffness of Specimens SC-2.4-0.20, SC-1.7-0.20, SC-2.4-0.50 and SC-1.7-0.50 obtained from the tests were 12.9 kN/mm, 26.9 kN/mm, 15.5 kN/mm and 34.4 kN/mm respectively. The increase in the initial stiffness between Specimens SC-1.7-0.20 and SC-2.4-0.20 was 108.5%. Similarly, an enhancement in the initial stiffenss of 121.9% was observed in Specimen SC-1.7-0.50 as compared to Specimen SC-2.4-0.50.

5.3.3 Shear Strength

Figure 5.7 plots the shear strength versus the column axial load ratio of all test specimens. The column axial load in the test specimen varied from 0.05 to 0.50 $f'_c A_g$. As observed in **Figure 5.7**, the shear strength of SC-1.7 Series specimens

enhanced by around 6.4%, 21.4%, and 35.9% as the column axial load was increased from 0.05 to 0.20, 0.35, and $0.50 f'_c A_g$, respectively. An analogous trend was observed in the specimens of RC-1.7 Series, whose shear strengths experienced an enhancement of around 7.9% and 22.1% with an increase in the column axial load from 0.05 to 0.20 and $0.35 f'_c A_g$, respectively. However, a slight increase of 2.7% in the shear strength was observed in RC-1.7 Series specimens, as the column axial load was increased from 0.35 to $0.50 f'_c A_g$. It is to be noted that Specimen RC-1.7-0.50 had the smallest ratio of theoretical flexural strength to nominal shear strength. For the specimens of SC-2.4 Series, an enhancement in the shear strength of around 8.5% was observed, as the column axial load was increased from 0.20 to $0.50 f'_c A_g$. The aforementioned discussion clearly indicates that the column axial load was beneficial to the shear strength of the test specimens.

Figure 5.7 Comparison of Shear Strength between Test Specimens

The shear strength of Specimens SC-2.4-0.20, SC-1.7-0.20, SC-2.4-0.50 and SC-1.7-0.50 obtained from the tests were 218.9 kN, 294.2 kN, 237.6 kN and 375.6 kN respectively. The increase in the shear strength between Specimens SC-2.4-0.20 and SC-1.7-0.20 was 34.4%. Similarly, an enhancement of 58.1% was observed in the

shear strength of Specimen SC-1.7-0.50 as compared to that of Specimen SC-2.4-0.50. Thus, it can be concluded that the shear strength of the test specimens was increased with a decrease of their aspect ratio.

Between the specimens of SC-1.7 and RC-1.7 Series, an increase in the shear strength of 2.4%, 3.8%, and 3.0% was recorded for the specimens with an axial load of 0.05, 0.20, and $0.35 f'_c A_g$ respectively. This could be attributed to the longer depth of RC-1.7 Series specimens as compared to SC-1.7 Series specimens. For the same shear crack angle, the longer the depth of the column is, the more the number of the transverse reinforcing bars crosses the shear crack, which leads to the higher transverse reinforcement contribution to the shear strength. In addition, both RC-1.7 Series and SC-1.7 Series specimens had the same cross sectional area and aspect ratio. Therefore, the same concrete contribution to the shear strength was expected in both RC-1.7 Series and SC-1.7 Series specimens. As compared with Specimen SC-1.7-0.50, Specimen RC-1.7-0.50 obtained the lower shear strength. As explained previously, the maximum shear force of Specimen RC-1.7-0.50 was controlled by the flexural strength, which then led to this result.

5.3.4 Drift Ratio at Axial Failure

Figure 5.8 shows the drift ratio at axial failure versus the column axial load ratio of the test specimens. The general trend of the curves in **Figure 5.8** showed that an increase in the column axial load ratio reduced the drift ratio at axial failure in all test series.

As observed in **Figure 5.8**, the drift ratio at axial failure in SC-1.7 and RC-1.7 Series specimens reduced sharply by around 83.9% and 74.6% respectively as the column axial load ratio was increased from 0.05 to 0.20. However, only a slight decrease of 14.3% and 29.6% in the drift ratio at axial failure was recorded in SC-1.7 and RC-1.7 Series specimens respectively, as the column axial load was increased from 0.20 to $0.35 f'_c A_g$. Further increasing the column axial load from 0.35 to $0.50 f'_c A_g$, similar trend was obtained in both SC-1.7 and RC-1.7 Series. An

analogous trend was observed in the specimens of SC-2.4 Series, whose drift ratios at axial failure experienced a reduction of around 40.4% with an increase in the column axial load from 0.20 to 0.50 $f'_c A_g$. Based on the aforementioned discussion, it is concluded that the column axial load had detrimental effects on the drift ratio at axial failure.

Figure 5.8 Comparison of Drift Ratio at Axial Failure between Test Specimens

The effects of aspect ratio on the drift ratio at axial failure can be noticed by comparing between SC-2.4 and SC-1.7 Series specimens. As shown in **Figure 5.8**, at a column axial load ratio of 0.20, the drift ratio at axial failure reduced from 2.82% to 1.82% with a decrease in the aspect ratio from 2.4 to 1.7. At a column axial load ratio of 0.50, a decrease of 20.8% was observed between the specimens from SC-2.4 and SC-1.7 Series. It can be concluded, based on the test results obtained from Specimens SC-2.4-0.20, SC-2.4-0.50, SC-1.7-0.20 and SC-1.7-0.50, that a decrease in their aspect ratio led to a reduction in the drift ratio at axial failure.

For an axial load ratio of 0.05, a slightly higher drift ratio at axial failure was

observed in the SC-1.7 as compared to RC-1.7 specimen. The higher drift ratio at axial failure of 57.6%, 29.5% and 26.8% was recorded in the specimen of RC-1.7 Series as compared to SC-1.7 Series for an axial load ratio of 0.20, 0.35 and 0.50 respectively. This observed trend was suggested to be due to the difference in the mode of axial failure of the test specimens. It is to be noted that both Specimen RC-1.7-0.05 and SC-1.7-0.05 shared the same mode of axial failure (Mode 2), where the axial failure in the specimens was attributed to the extended damaged zone. While for an axial load ratio of 0.20 and 0.50, different modes of axial failure were observed in the specimens of RC-1.7 and SC-1.7 Series.

Table 5.1 Comparisons between Test Specimens

Specimen	Initial Stiffness (kN/mm)	Shear Strength (kN)	Drift Ratio at Axial Failure (%)	Maximum Cumulative Energy Dissipation (kNm)
SC-2.4-0.20	12.9	218.9	2.82	34.9
SC-2.4-0.50	15.5	237.6	1.68	26.3
SC-1.7-0.05	24.5	276.4	11.29	35.1
SC-1.7-0.20	26.9	294.2	1.82	13.5
SC-1.7-0.35	28.8	335.5	1.56	9.1
SC-1.7-0.50	34.4	375.6	1.42	4.2
RC-1.7-0.05	11.5	283.1	11.30	77.1
RC-1.7-0.20	15.4	305.5	2.87	44.3
RC-1.7-0.35	18.9	345.7	2.02	26.5
RC-1.7-0.50	21.4	355.2	1.80	23.6

On comparing the drift ratio at axial failure between specimens in RC-1.7 and SC-2.4 Series, it was observed that between Specimen RC-1.7-0.20 and SC-2.4-0.20, there was a slight reduction in the drift ratio at axial failure by approximately 1.8% (from 2.87% to 2.82%). Similarly, between Specimen RC-1.7-0.50 and SC-2.4-0.50, there was a slight reduction in the drift ratio at axial failure by approximately 7.1% (from 1.80% to 1.68%).

5.4 Energy Dissipation

In this section, the seismic performance of the test specimens is further compared through the comparison between the cumulative energy dissipation obtained from the test specimens. **Figures 5.9, 5.10 and 5.11** show the comparison between the cumulative energy dissipation obtained from the test specimens in SC-2.4, SC-1.7, and RC-1.7 Series; respectively. **Figure 5.12** plots the maximum cumulative energy dissipation versus the column axial load ratio of all test specimens.

As illustrated in **Figure 5.12**, the maximum cumulative energy dissipation in SC-1.7 and RC-1.7 Series specimens reduced sharply by around 61.5% and 42.5% respectively as the column axial load ratio was increased from 0.05 to 0.20. However, only a slight decrease in the maximum cumulative energy dissipation was recorded in SC-1.7 and RC-1.7 Series specimens as the column axial load was increased from 0.20 to $0.35 f'_c A_g$. Further increasing the column axial load from 0.35 to $0.50 f'_c A_g$, similar trend was obtained in both SC-1.7 and RC-1.7 Series.

An analogous trend was observed in the specimens of SC-2.4 Series, whose maximum cumulative energy dissipation experienced a reduction of around 24.6% with an increase in the column axial load from 0.20 to $0.50 f'_c A_g$. Based on the aforementioned discussion, it can be concluded that the column axial load had detrimental effects on the maximum cumulative energy dissipation.

The effects of aspect ratio on the maximum cumulative energy dissipation can be noticed by comparing between SC-2.4 and SC-1.7 Series specimens. As shown in **Figure 5.12**, at the column axial load ratio of 0.20, the maximum cumulative energy dissipation reduced from 34.9 kN/mm to 13.5 kN/mm as its aspect ratio decreases from 2.4 to 1.7. At the column axial load ratio of 0.50, a decrease of 84.0% was observed between the specimen of SC-2.4 and SC-1.7 Series. Based on the test results of Specimens SC-2.4-0.20, SC-2.4-0.50, SC-1.7-0.20 and SC-1.7-0.50, it can be concluded that a decrease in the aspect ratio also decreased the maximum cumulative energy dissipation.

On comparing the maximum cumulative energy dissipation between specimens in RC-1.7 and SC-1.7 Series, it was observed that the maximum cumulative energy dissipation obtained in the specimen of RC-1.7 Series was higher than that of SC-1.7 Series for all axial load ratios.

Figure 5.9 Cumulative Energy Dissipation of SC-2.4 Series Specimens

Figure 5.10 Cumulative Energy Dissipation of SC-1.7 Series Specimens

Figure 5.11 Cumulative Energy Dissipation of Specimens RC-1.7 Series

Figure 5.12 Comparison of Maximum Cumulative Energy Dissipation between Test Specimens

5.5 Comparison with Seismic Assessment Models

In this section, the backbone curves obtained from the experimental results of all test specimens are compared with FEMA 356 [F1] and ASCE 41 [E8]'s models. According to FEMA 356 [F1] and ASCE 41 [E8], the force-displacement relationship follows the general trend as shown in **Figure 5.13**.

Figure 5.13 Generalized Force-Displacement Relationship in FEMA 356 [F1] and ASCE 41 [E8]

Table 5.2 Flexural Rigidity in FEMA 356 [F1] and ASCE [E8]

	FEMA 356 [F1]	ASCE [A1]
$P \geq 0.5 f'_c A_g$	$0.7 E_c I_g$	$0.7 E_c I_g$
$P \leq 0.3 f'_c A_g$	$0.5 E_c I_g$	—
$P \leq 0.1 f'_c A_g$	—	$0.3 E_c I_g$

Note: Linear interpolation between values listed in the table shall be permitted

Flexural rigidity as well as shear rigidity are considered in calculating the initial stiffness of columns in both FEMA 356 [F1] and ASCE 41 [E8]. Shear rigidity for rectangular cross sections is defined as $0.4 E_c A_g$ in both FEMA 356 [F1] and ASCE 41 [E8]. According to FEMA 356 [F1] and ASCE 41 [E8], flexural rigidity is related to applied column axial loads as tabulated in **Table 5.2**.

The deformation indexes (a , b) as illustrated in **Figure 5.13** are defined as flexural plastic hinge ratios which depend on column axial load, nominal shear stress and details of columns. The index c as defined in FEMA 356 [F1] is equal to 0.2, whereas this index based on ASCE 41 [E8] is in a range of zero to 0.2, which depend on column axial load, nominal shear stress and details of columns. **Table 5.3** summarizes all the indexes (a , b , c) of all the test specimens calculated based on FEMA 356 [F1] and ASCE 41 [E8].

Table 5.3 Modelling Parameters

Specimen	FEMA 356 [F1]			ASCE 41 [E8]			
	a	b	c	Condition	a	b	c
SC-2.4-0.20	0.0043	0.0114	0.2	iii	0	0.0113	0
SC-2.4-0.50	0.0021	0.0082	0.2	iii	0	0.0037	0
SC-1.7-0.05	0.0050	0.0120	0.2	iii	0	0.0139	0
SC-1.7-0.20	0.0040	0.0107	0.2	iii	0	0.0113	0
SC-1.7-0.35	0.0025	0.0087	0.2	iii	0	0.0075	0
SC-1.7-0.50	0.0020	0.0080	0.2	iii	0	0.0037	0
RC-1.7-0.05	0.0052	0.0125	0.2	iii	0	0.0188	0
RC-1.7-0.20	0.0040	0.0107	0.2	iii	0	0.0154	0
RC-1.7-0.35	0.0025	0.0087	0.2	ii	0.0062	0.0104	0.0470
RC-1.7-0.50	0.0020	0.0080	0.2	ii	0.0036	0.0053	0.0470

Note: Condition iii = Shear Failure; Condition ii = Shear-flexure Failure. [E8]

According to both FEMA 356 [F1] and ASCE 41 [E8] guidelines, the maximum shear force of the column is limited by its shear strength. Where the shear strength as defined in both FEMA 356 [F1] and ASCE 41 [E8] is given as:

$$V_n = k_1 \frac{A_v f_y d}{s} + \lambda k_2 \left(\frac{0.5 \sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{0.5 \sqrt{f'_c} A_g}} \right) 0.8 A_g \text{ (MPa)} \quad (5.1)$$

where k_1 is equal to 1 for transverse steel spacing less than or equal to $d/2$, k_1 is equal to 0.5 for spacing exceeding $d/2$ but not more than d , k_1 is equal to 0 otherwise; k_2 is taken as 1 for displacement ductility less than 2, as 0.7 for displacement ductility more than 4 and varies linearly for intermediate displacement ductility; a/d shall not be taken greater than 3 or less than 2; and λ is equal to 1 for normal-weight concrete.

Figure 5.14 compares the backbone curves of the test specimens with analytical results obtained from FEMA 356 [F1] and ASCE 41 [E8]'s models. The test results showed that both FEMA 356 [F1] and ASCE 41 [E8] guidelines provided a good prediction of the shear strength of the test specimens. However, the column initial stiffness and ultimate displacements (displacements at axial failure) were over-estimated and under-estimated by FEMA 356 [F1] and ASCE 41 [E8] guidelines, respectively.

(a)

(b)

(c)

(d)

(e)

(f)

(g)

(h)

(i)

(j)

Figure 5.14 Comparison between Experimental Backbone Curves and FEMA 356 [F1] and ASCE 41 [E8] 's Models

5.6 Summary

This chapter compared the experimental results from all test specimens in terms of cracking patterns, backbone curves and energy dissipation. The backbone curves obtained from experimental results of the test specimens were also compared with FEMA 356 [F1] and ASCE 41 [E8] models. Based on the above comparisons as well as the test results reported in **Chapter 4**, the following conclusions can be drawn:

1. There were two distinctive shear failure modes observed in the test specimens. In the first mode, the failure was controlled by a steep shear crack, whereas shear and bond-splitting cracks controlled the failure in the second mode.
2. There were two axial failure modes observed in the test specimens. In the first mode, the steep shear crack developed on the column from the previous stages became wider. This led to sliding between the cracking surfaces as well as the buckling of longitudinal reinforcing bars and fracturing of transverse reinforcing bars along this shear crack. In the second mode, crushing of concrete as well as the buckling of longitudinal reinforcing bars and fracturing of transverse were developed across a damaged zone.
3. The column axial load had a detrimental effect on the drift ratio at axial failure and maximum energy dissipation capacity of the test specimens. However, the shear strength and initial stiffness increased with an increase in column axial load.
4. The drift ratio at axial failure and maximum energy dissipation capacity of test specimens dropped with a decrease in aspect ratio. However, the shear strength and initial stiffness increased with a decrease in aspect ratio.

5. Better seismic performances were observed in RC-1.7 Series specimens as compared to SC-2.4 and SC-1.7 Series specimens at the same column axial load.
6. A decrease in aspect ratio and an increase in column axial load have led to the predominance of shear over flexural deformations. This is a clear sign of the transition from a flexural to shear mode of failure, which subsequently leads to a reduction in the drift ratio at axial failure in the test specimens.
7. The test results showed that both FEMA 356 [F1] and ASCE 41 [E8] guidelines provided a good prediction of the shear strength of the test specimens. However, the column initial stiffness and ultimate displacements (displacements at axial failure) were over-estimated and under-estimated by both FEMA 356 [F1] and ASCE 41 [E8] guidelines, respectively. Further research works are needed to accurately capture the initial stiffness and the ultimate displacements of the test specimens.

CHAPTER 6

INITIAL STIFFNESS OF REINFORCED CONCRETE COLUMNS WITH MODERATE ASPECT RATIOS

6.1 Introduction

In recent years, the earthquake design philosophy has been shifted from a more traditional force-based approach towards a displacement-based ideology. The assumed initial stiffness of columns could affect the estimation of the displacement and displacement ductility, which are crucial in the displacement-based design. In addition, the assumed initial stiffness properties of columns also affect the estimation of the fundamental period and distribution of internal forces in whole structures. Therefore, an accurate evaluation of the initial stiffness of columns becomes an inevitable requirement.

Literature reviews show that there is a considerable amount of uncertainty regarding to the estimation of initial stiffness of columns when subjected to seismic loading. Current design codes often employ a stiffness reduction factor to deal with this uncertainty. In an attempt to address these uncertainties, the study presented within this chapter was devoted to developing a rational method to determine the initial stiffness of RC columns when subjected to seismic loads. A comprehensive parametric study based on the proposed method was carried out to investigate the influences of several critical factors. A simple equation to estimate the initial stiffness of RC columns is also proposed within this chapter. The applicability and accuracy of the proposed method and equation are then verified with the experimental data presented in **Chapter 4** and in the literature.

This chapter reported herein comprises three parts. The first part is devoted to review the existing guidelines to calculate the initial stiffness of columns. In the second part, the proposed method to estimate the initial stiffness of columns is

presented. The parametric study based on the proposed method is carried out in the final part of the chapter.

6.2 Review of Existing Initial Stiffness Models

In the following sections, the effective moment of inertia (I_e) is defined as the moment of inertia that a uniform elastically responding columns would have, such that when it is subjected to the lateral force that causes first yield, or a strain of 0.002 in the concrete, it sustains the same deflection. This definition of the effective moment of inertia is utilized in several previous researchers and design codes for estimating initial stiffness of RC members. They are briefly reviewed in the subsequent sections.

6.2.1 ACI 318-08 [A1]

ACI 318-08 [A1] recommends the following options for estimating EI_e for the determination of lateral deflection of building systems subjected to factored lateral loads: (a) $0.35 EI_g$ for members with an axial load ratio of less than 0.10 and $0.70 EI_g$ for members with an axial load ratio of more than or equal to 0.10; or (b) $0.50 EI_g$ for all members.

6.2.2 FEMA 356 [F1]

FEMA 356 [F1] suggests the variation of EI_e values with the applied axial load ratio. EI_e is taken as $0.50 EI_g$ for members with an axial load ratio of less than 0.30, as $0.7 EI_g$ for members with an axial load ratio of more than 0.50 and varies linearly for intermediate axial load ratios as illustrated in **Figure 6.1**.

6.2.3 ASCE 41 [A2]

As shown in **Figure 6.1**, ASCE 41 [A2] recommends that EI_e is taken as $0.30 EI_g$ for members with an axial load ratio of less than 0.10, as $0.7 EI_g$ for members with an axial load ratio of more than 0.50 and varies linearly for intermediate axial load ratios.

6.2.4 Paulay and Priestley [P2]

According to Paulay and Priestley's recommendation [P2], EI_e is taken as $0.40 EI_g$ for members with an axial load ratio of less than -0.05, as $0.8 EI_g$ for members with an axial load ratio of more than 0.50 and varies linearly for intermediate axial load ratios as illustrated in **Figure 6.1**.

Figure 6.1 Relationships between Stiffness Ratio and Axial Load Ratio of Existing Models

6.2.5 Elwood and Eberhard [E7]

EI_e is taken as kEI_g based on Elwood and Eberhard's recommendation [E7]. The stiffness ratio k is defined by the following equation:

$$k = \frac{0.45 + 2.5P / A_g f'_c}{1 + 110 \left(\frac{d_b}{h} \right) \left(\frac{h}{a} \right)} \leq 1 \text{ and } \geq 0.2 \quad (6.1)$$

where d_b is the diameter of longitudinal reinforcing bars; a is the shear span and h is the column depth.

Figure 6.1 illustrates the variation of stiffness ratio based on Elwood and Eberhard's model [E7] versus the axial load ratio for specimens with d_b and a equal to 25 mm and 850 mm respectively.

6.3 Defining Initial Stiffness for RC Columns

There are two methods as illustrated in **Figure 6.2(a)** to determine the initial stiffness of RC columns. In the first method, the initial stiffness of RC columns are estimated by using the secant of the shear force versus its lateral displacement relationship passing through the point at which the applied force reaches 75% of the flexural strength ($0.75 V_u$). In the second method, the column is loaded until either the first yield occurs in the longitudinal reinforcement or the maximum compressive strain of concrete reaches 0.002 at a critical section of the column. This corresponds to point A in **Figure 6.2(a)**. Generally, the two approaches give similar values. In this study, the later approach was adopted.

However, the above-mentioned definition can not be used for columns with shear strength do not substantially exceed its theoretical yield force. For these columns, defined as those whose maximum measured shear force was less than 107% of the theoretical yield force, the initial stiffness was defined based on a point on the measured force-displacement envelope with a shear force equal to $0.8 V_{\max}$ as illustrated in **Figure 6.2 (b)** [E7].

Figure 6.2 Methods to Determine Initial Stiffness [E7]

6.4 Proposed Method to Estimate Initial Stiffness of RC Columns

6.4.1 Yield Force (V_y)

The initial stiffness of columns is determined by applying the second method as described in the previous section. The yield force (V_y) corresponding to point A in **Figure 6.2** is obtained from the yield moment (M_y) when the reinforcing bar closest to the tension edge of columns has reached its yield strain. Moment-curvature analysis is adopted to determine this moment.

6.4.2 Displacement at Yield Force (Δ'_y)

The displacement of a column at yield force (V_y) can be considered as the sum of the displacement due to flexure, bar slip and shear.

$$\Delta'_y = \Delta'_{flex} + \Delta'_{shear} \quad (6.2)$$

where Δ'_y is the displacement of a column at yield force; Δ'_{flex} is the displacement due to flexure and bar slip at yield force; and Δ'_{shear} is the displacement due to shear at yield force

Flexure Deformations (Δ'_{flex})

In this proposed method, the simplified concept of an effective length of the member suggested by Priestley *et al.* [P4] was used to account for the displacement due to bar slip in flexure deformations. Assuming a linear variation in curvature over the height of the column, the contribution of flexural deformations and bar slips to the displacement at the yield force for RC columns with a fixed condition at both ends can be estimated as follows:

$$\Delta'_{flex} = \frac{\phi'_y (L + 2L_{sp})^2}{6} \quad (6.3)$$

where ϕ'_y is the curvature at the yield force determined by using moment-curvature analysis and L is the clear height of columns.

The strain penetration length (L_{sp}) is given by:

$$L_{sp} = 0.022 f_{yl} d_b \quad (6.4)$$

where f_{yl} is the yield strength of longitudinal reinforcing bars; and d_b is the diameter of longitudinal reinforcing bars.

Shear Deformations (Δ'_{shear})

The idea of utilizing the truss analogy to model cracked RC elements has been around for many years. The truss analogy is a discrete modeling of actual stress fields within RC members. The complex stress fields within structural components resulting from applied external forces are simplified into discrete compressive and tensile load paths. The analogy utilizes the general idea of concrete in compression and steel reinforcement in tension. The longitudinal reinforcement in a beam or column represents the tensile chord of a truss while the concrete in the flexural compression zone is considered as part of the longitudinal compressive chord. The transverse reinforcement serves as ties holding the longitudinal chords together. The diagonal concrete compression struts, which discretely simulate the concrete compressive stress field, are connected to the ties and longitudinal chords at rigid nodes to attain static equilibrium within the truss. The truss analogy is a very promising way to treat shear because it provides a visible representation of how forces are transferred in a RC members under an applied shear force.

Park and Paulay [P1] derived a method to determine the shear stiffness by applying the truss analogy, for short or deep rectangular beams of unit length. The shear stiffness is the magnitude of the shear force, when applied to a beam of unit length that will cause unit shear displacement at one end of the beam relative to the other. This model is reliable in estimating shear deformations of short or deep beams in which the influences of flexure are negligible. The behaviors of RC columns under seismic loading are much more complex because of the interaction between shear and flexure. The influences of axial strain due to flexure in estimating shear deformations of RC columns should be considered to accurately predict the initial stiffness of RC columns. By applying a method that is similar to Park and Paulay's analogous truss model [P1], the shear stiffness of RC columns is derived in this part of the chapter. The effects of flexure in shear deformations are incorporated in the

proposed model through the axial strains at the center of columns ($\varepsilon_{y,CL}$).

Assuming that transverse reinforcing bars start resisting the applied shear force when the shear cracking starts occurring, the stress in transverse reinforcing bars at the yield force is calculated as:

$$f_{sy} = \frac{(V_y - V_{cr})s}{A_{st}d \tan \theta} \quad (6.5)$$

where d is the distance from the extreme compression fiber to centroid of tension reinforcement; s is the spacing of transverse reinforcement; A_{st} is the total transverse steel area within spacing s ; and θ is the angle of diagonal compression strut. Hence the strain in transverse reinforcing bars is:

$$\varepsilon_x = \frac{f_{sy}}{E_s} \leq \varepsilon_{yt} \quad (6.6)$$

where ε_{yt} is the yield strain of transverse reinforcing bars; E_s is the elastic modulus of steel.

Similar to Park and Paulay's model [P1], the concrete compression stress at the yield force is given as:

$$f_2 = \frac{V_y}{bL_{cs} \cos \theta} \quad (6.7)$$

where b is the width of columns; $L_{cs} = d \sin \theta$ is the effective depth of the diagonal strut as shown in **Figure 6.3**.

Figure 6.3 Diagonal Strut of RC Columns [P1]

Hence the strain in the concrete compression strut is given as:

$$\varepsilon_2 = \frac{f_2}{E_c} \quad (6.8)$$

where E_c is the elastic modulus of concrete [P5] given as:

$$E_c = 5000\sqrt{f_c} \quad (6.9)$$

Based on Vecchio and Collins's model [V1], the effective compressive strength of concrete is calculated as follows:

$$f_{ce} = \frac{f'_c}{0.8 + 170\varepsilon_1} \leq f'_c \quad (6.10)$$

By applying Mohr's circle transformation for the mean strains at the center of **Section C-C** as shown in **Figure 6.4**, it gives:

$$\varepsilon_1 = \frac{\varepsilon_x + \varepsilon_{y,CL}}{2} + \sqrt{\left(\frac{\varepsilon_x - \varepsilon_{y,CL}}{2}\right)^2 + \left(\frac{\gamma_{xy}}{2}\right)^2} \quad (6.11)$$

$$\varepsilon_2 = \frac{\varepsilon_x + \varepsilon_{y,CL}}{2} - \sqrt{\left(\frac{\varepsilon_x - \varepsilon_{y,CL}}{2}\right)^2 + \left(\frac{\gamma_{xy}}{2}\right)^2} \quad (6.12)$$

$$\tan 2\theta = \frac{\gamma_{xy}}{\epsilon_x - \epsilon_{y,CL}} \quad (6.13)$$

Figure 6.4 Influences of Flexure in Estimating Shear Deformations

For the axial mean strains, compatibility requires that the plain sections remain plane. Hence the mean strain at the center of section C-C is given as:

$$\epsilon_{y,CL} = \frac{\epsilon_{y,top} + \epsilon_{y,bot}}{2} \quad (6.14)$$

where $\epsilon_{y,top}$, $\epsilon_{y,bot}$ are the axial strains at the extreme tension and compression

fibers, respectively as shown in **Figure 6.4(b)**.

There are six variables, namely ε_x , $\varepsilon_{y,CL}$, γ_{xy} , ε_1 , ε_2 and θ ; and six independent equations (6.6), (6.8), (6.9), (6.10), (6.11) and (6.12). By solving these six independent equations, the shear strain (γ_{xy}) at the center of section C-C could be determined.

The column is divided into several segments along its height to determine the total shear deformation at the top of the column. The mean axial strain at the center of the section is determined based on the moment-curvature analysis. The shear strains at the lower and upper section of the segment are calculated using the above equations. Hence, the total shear displacement caused by the yield force can be calculated as follows:

$$\Delta'_{shear} = \sum_{i=1}^n \left(\frac{\gamma_{xy}^i + \gamma_{xy}^{i+1}}{2} \right) h_i \quad (6.15)$$

where γ_{xy}^i and γ_{xy}^{i+1} are the shear strains at the lower and upper section of the segment i ; h_i is the height of segment i and n is the number of segments.

6.4.3 Initial Stiffness

Once the flexural and shear deformations at the top of columns under yield force are obtained, the initial stiffness of columns can be determined as:

$$K_i = \frac{V_y}{\Delta'_{flex} + \Delta'_{shear}} \quad (6.16)$$

Hence the effective moment of inertia for a column can be expressed as:

$$I_e = \frac{L^3 K_i}{12 E_c} \quad (6.17)$$

The stiffness ratio (k) is defined as follows:

$$k = \frac{EI_e}{EI_g} \times 100\% \quad (6.18)$$

where I_g is the moment of inertia of the gross section.

6.5 Validation of the Proposed Method

The proposed method is validated by comparing its results to the initial stiffness of ten RC columns obtained from the current experimental study.

Table 6.1 Experimental Verification of the Proposed Method

Specimen	$K_{i-\text{exp}}$ (kN/mm)	$\frac{K_{i-\text{exp}}}{K_{i-p}}$	$\frac{K_{i-\text{exp}}}{K_{i-ACI(a)}}$	$\frac{K_{i-\text{exp}}}{K_{i-ACI(b)}}$	$\frac{K_{i-\text{exp}}}{K_{i-FEMA}}$	$\frac{K_{i-\text{exp}}}{K_{i-ASCE}}$	$\frac{K_{i-\text{exp}}}{K_{i-PP}}$	$\frac{K_{i-\text{exp}}}{K_{i-EE}}$
SC-2.4-0.20	12.9	0.782	0.254	0.355	0.355	0.444	0.305	0.793
SC-2.4-0.50	15.5	0.572	0.301	0.421	0.301	0.301	0.263	0.525
SC-1.7-0.05	24.5	0.918	0.319	0.223	0.223	0.372	0.236	0.560
SC-1.7-0.20	26.9	0.865	0.169	0.236	0.236	0.295	0.203	0.590
SC-1.7-0.35	28.8	0.653	0.188	0.263	0.239	0.239	0.190	0.553
SC-1.7-0.50	34.4	0.620	0.220	0.308	0.220	0.220	0.193	0.507
RC-1.7-0.05	11.5	0.898	0.208	0.145	0.145	0.242	0.154	0.365
RC-1.7-0.20	15.4	0.846	0.140	0.196	0.196	0.244	0.168	0.442
RC-1.7-0.35	18.9	0.661	0.173	0.243	0.221	0.221	0.176	0.391
RC-1.7-0.50	21.4	0.583	0.197	0.276	0.197	0.197	0.173	0.348
Mean		0.740	0.217	0.267	0.233	0.278	0.206	0.507
Coefficient of Variation		0.136	0.058	0.079	0.058	0.078	0.048	0.132

It was found that the average ratio of experimental to predicted initial stiffness by the proposed method was 0.740 as tabulated in **Table 6.1**. This shows a relatively good correlation between the analytical and experimental results. The initial stiffness of test columns calculated based on ACI 318-2008 [A1], FEMA 356 [F1], ASCE 41 [A2], Paulay and Priestley [P2], and Elwood and Eberhard [E7] are also

tabulated in **Table 6.1**. The mean ratio of the experimental to predicted initial stiffness and its coefficient of variation were 0.217 and 0.058 for ACI 318-2008 (a) [A1], 0.267 and 0.079 for ACI 318-2008 (b) [A1], 0.233 and 0.058 for FEMA 356 [F1], 0.278 and 0.078 for ASCE 41 [A2], 0.206 and 0.048 for Paulay and Priestley [P2], and 0.507 and 0.132 for Elwood and Eberhard [E7], respectively. Comparison of available models with experimental data indicated that the proposed method produced a better mean ratio of the experimental to predicted initial stiffness than other models. The proposed method may be suitable as an assessment tool to calculate the initial stiffness of RC columns.

6.6 Parametric Study

A parametric study conducted to improve the understanding of the effects of various parameters on the initial stiffness of RC columns is presented within this section. The parameters investigated are transverse reinforcement ratios (ρ_v), longitudinal reinforcement ratios (ρ_l), yield strength of longitudinal reinforcing bars (f_{yl}), concrete compressive strength (f'_c), aspect ratio (a/d) and axial load ratio ($P/f'_c A_g$). In the parametric study, the effects of parameters that were investigated on the initial stiffness of RC columns are presented by the dimensionless stiffness ratio (k).

Specimen SC-2.4-0.20, with an aspect ratio of 2.4 as presented in Chapter 3 is considered as the reference specimen in the parametric study. An axial load of $0.2 f'_c A_g$ was applied to the specimen. The concrete compressive strength of the specimen (f'_c) at 28 days was 25.0 MPa. The longitudinal reinforcement consisted of 8-T20 (20 mm diameter). This resulted in the ratio of longitudinal steel area to the gross area of column to be 2.05%. The transverse reinforcement consisted of R6 bars (6 mm diameter) with 135° bent spaced at 125 mm, corresponding to a transverse reinforcement ratio of 0.129%.

6.6.1 Influence of Transverse Reinforcement Ratio

The analyses as illustrated in **Figure 6.5** were conducted to assess the influence of transverse reinforcement on the effective moment of inertia. Two column axial loads of $0.05 f'_c A_g$ and $0.20 f'_c A_g$ were considered. Five types of transverse reinforcement, R6-125 mm, R8-125 mm, R8-100mm, R10-125mm and R10-100 which correspond to five transverse reinforcement ratios ρ_v of 0.129%, 0.230%, 0.287%, 0.359% and 0.449% respectively, were investigated.

Figure 6.5 Influences of Transverse Reinforcement Ratios on Stiffness Ratio

Figure 6.5 shows that with an increase in transverse reinforcement content from 0.129% to 0.230%, 0.287%, 0.359% and 0.449%, stiffness ratios rose slightly by approximately 3.4%, 4.5%, 5.5%, 6.4%, respectively for columns under an axial load of $0.20 f'_c A_g$. The stiffness ratios increased by approximately 2.3%, 3.6%, 4.9%, 6.1% for columns under an axial load of $0.05 f'_c A_g$ with an increase in transverse reinforcement content from 0.129% to 0.230%, 0.287%, 0.359% and 0.449%, respectively. This suggested that the effect of transverse reinforcement ratios on stiffness ratios is insignificant. In addition, **Figure 7.5** shows a clear indication that stiffness ratio increases with an increase in column axial load.

6.6.2 Influence of Longitudinal Reinforcement Ratio

The influence of longitudinal reinforcement ratios on stiffness ratios is presented in **Figure 6.6** for two different column axial loads of $0.05 f'_c A_g$ and $0.20 f'_c A_g$. Four types of longitudinal reinforcement, 8T16, 8T20, 8T22 and 8T25 corresponding to longitudinal reinforcement ratios ρ_l of 1.66%, 2.05%, 2.48% and 3.21% respectively, were considered.

Figure 6.6 Influences of Longitudinal Reinforcement Ratio on Stiffness Ratio

As shown in **Figure 6.6**, the stiffness ratios for columns under an axial load of $0.05 f'_c A_g$ were observed to rise slightly with an increase in longitudinal reinforcement ratio; while for columns under an axial load of $0.20 f'_c A_g$ the stiffness ratios almost remained the same. This suggested that for simplicity the influence of longitudinal reinforcement ratio on the initial stiffness of RC columns could be ignored.

6.6.3 Influence of Yield Strength of Longitudinal Reinforcing Bars

Four yield strengths of longitudinal reinforcing bars, 362MPa, 412MPa, 462MPa

and 512MPa were chosen to investigate the influences of this variable on stiffness ratios. As shown in **Figure 6.7**, with a decrease in yield strength of longitudinal reinforcing bars from 512MPa to 462MPa, 412MPa and 362MPa; the stiffness ratios increased slightly by approximately 3.1%, 4.3%, and 5.0%, respectively for columns under an axial load of $0.05 f'_c A_g$; whereas stiffness ratios almost remains the same for column under an axial load of $0.20 f'_c A_g$. The analytical results suggested that the influences of yield strength of longitudinal reinforcing bars on stiffness ratios are negligible.

Figure 6.7 Influences of Yield Strength of Longitudinal Reinforcing Bars on Stiffness Ratio

6.6.4 Influence of Concrete Compressive Strength

Figure 6.8 illustrates the influence of concrete compressive strength on stiffness ratios for two different axial loads of $0.05 f'_c A_g$ and $0.20 f'_c A_g$. The concrete compressive strengths investigated were 25MPa, 35MPa, 45MPa, and 55MPa. For both axial loads, with an increase in concrete compressive strength, no significant changes on stiffness ratios were observed.

Figure 6.8 Influences of Concrete Compressive Strength on Stiffness Ratio

6.6.5 Influence of Aspect Ratio

Figure 6.9 and **Table 6.2** show the influence of aspect ratio on stiffness ratios of RC columns. Six aspect ratios of 1.50, 1.80, 2.10, 2.43, 2.70, and 3.00 were investigated. In general, the stiffness ratio increased with an increase in aspect ratio.

Figure 6.9 shows that with an increase in aspect ratio from 1.50 to 1.80, 2.10, 2.43, 2.70, and 3.00; the stiffness ratios of columns without axial loads rose by approximately 18.5%, 39.8%, 62.8%, 83.6%, 109.4%, respectively. Similar trends were observed for the columns with an axial load ratio of 0.20. The stiffness ratios increased by approximately 15.6%, 27.4%, 37.8%, 45.2% and 52.3% for columns under an axial load of $0.60 f'_c A_g$ with an increase in aspect ratio from 1.50 to 1.80, 2.10, 2.43, 2.70, and 3.00, respectively. This suggested that the aspect ratio significantly influences the stiffness ratio.

Figure 6.9 Influences of Aspect Ratio on Stiffness Ratio

6.6.6 Influence of Axial Load

It is generally recognized that the presence of column axial load can effectively increase the flexural strength of columns and thus lead to larger initial flexural stiffness, which results in a higher stiffness ratio. The analyses as illustrated in **Figure 6.10** and tabulated in **Table 6.2** were carried out to assess the influence of axial load ratio on the stiffness ratio. The axial load ratio was varied from 0 to 0.60.

In general, the stiffness ratio increased with an increase in axial load ratio. **Figure 6.10** showed that with an increase in axial load ratio from 0 to 0.20, 0.40, and 0.60; the stiffness ratios for specimens with an aspect ratio of 1.5 rose by approximately 35.2%, 98.7% and 167.9%, respectively. Similar trends were observed for other aspect ratios. It can thus be concluded that the axial load ratio significantly affects the stiffness ratio.

Figure 6.10 Influences of Axial Load Ratio on Stiffness Ratio

Table 6.2 Stiffness Ratio for Various Aspect Ratios and Axial Load Ratios

	a/h					
$P/f'_c A_g$	1.50	1.80	2.10	2.43	2.70	3.00
0.00	11.22	13.30	15.69	18.27	20.60	23.50
0.05	12.27	14.24	16.64	19.24	21.13	23.90
0.10	13.32	15.45	17.78	20.23	22.21	24.20
0.15	14.23	16.54	18.85	21.46	23.37	25.27
0.20	15.17	17.66	20.13	22.83	24.80	26.70
0.25	16.43	19.23	22.56	25.61	27.75	29.76
0.30	17.90	21.83	25.70	29.06	31.30	33.22
0.35	19.78	24.85	28.77	31.91	33.85	35.50
0.40	22.30	27.57	31.27	34.22	36.05	37.73
0.45	24.74	29.70	33.27	36.12	38.01	39.81
0.50	26.82	31.73	35.28	38.14	40.16	42.08
0.55	28.56	33.37	36.82	39.86	41.94	43.95
0.60	30.06	34.74	38.30	41.42	43.66	45.77

6.7 Proposed Equation for Effective Moment of Inertia of RC Columns

It is observed that the stiffness ratio apparently increased with an increase in aspect ratios (R_a) and axial load ratio (R_n). The transverse and longitudinal reinforcement ratios, yield strength of longitudinal bars and concrete compressive strength insignificantly influenced the stiffness ratio of RC columns. For simplicity, the influences of these factors were ignored. Based on the results of the parametric study, the stiffness ratio (k) is given by the following equation:

$$k = (2.043R_n^2 + 2.961R_n + 1.739)(3.023R_a + 2.573) \quad (6.19)$$

Berry *et al.* [B2] collected a database of 400 tests of RC columns, which contained the hysteretic response, geometry, column axial load and material properties of test specimens. This database provided the data needed to evaluate the accuracy of the proposed equation for the stiffness ratio. The verification was limited to the range of the parametric study. The axial load was limited from 0 to $0.60 f'_c A_g$, and the aspect ratio was limited from 1.5 to 3.0. Only rectangular columns tested in the double-curvature configuration under unidirectional quasi-static cyclic lateral loading were chosen. Details of the chosen RC columns are tabulated in **Table 6.3**.

It was found that the average ratio of the experimental to predicted stiffness ratio by the proposed equation is 0.897 as shown in **Table 6.3**, showing a good correlation between the proposed equation and experimental data. Therefore, the proposed equation may be suitable as an assessment tool to calculate the stiffness ratio of RC columns within the range of the parametric study.

The stiffness ratio of columns calculated based on ACI 318-2008 [A1], FEMA 356 [F1], ASCE 41 [A2], and Paulay and Priestley [P2] are also shown in **Table 6.3**. The mean ratio of the experimental to predicted stiffness ratio and its coefficient of variation were 0.382 and 0.147 for ACI 318-2008 (a) [A1], 0.388 and 0.110 for ACI 318-2008 (b) [A1], 0.337 and 0.113 for FEMA 356 [F1], 0.534 and 0.180 for ASCE 41 [A2], 0.357 and 0.112 for Paulay and Priestley [P2], and 0.804 and 0.241 for

Elwood and Eberhard [E7], respectively. Comparison of available models with experimental data indicated that the proposed equation produced a better mean ratio of the experimental to predicted stiffness ratio than other models.

Table 6.3 Experimental Verification of the Proposed Equation

	Specimen	R_a	R_n	k_p (%)	k_{exp} (%)	$\frac{k_{exp}}{k_p}$	$\frac{k_{exp}}{k_{ACI(a)}}$	$\frac{k_{exp}}{k_{ACI(b)}}$	$\frac{k_{exp}}{k_{FEMA}}$	$\frac{k_{exp}}{k_{ASCE}}$	$\frac{k_{exp}}{k_{PP}}$	$\frac{k_{exp}}{k_{EE}}$
Current Experiment	SC-2.4-0.20	2.43	0.200	23.9	17.8	0.745	0.254	0.355	0.355	0.444	0.305	0.793
	SC-2.4-0.50	2.43	0.500	37.0	21.1	0.570	0.301	0.421	0.301	0.301	0.263	0.525
	SC-1.7-0.05	1.71	0.050	14.6	11.2	0.767	0.319	0.223	0.223	0.372	0.236	0.560
	SC-1.7-0.20	1.71	0.200	18.7	11.8	0.631	0.169	0.236	0.236	0.295	0.203	0.590
	SC-1.7-0.35	1.71	0.350	23.4	13.1	0.560	0.188	0.263	0.239	0.239	0.190	0.553
	SC-1.7-0.50	1.71	0.500	28.9	15.4	0.533	0.220	0.308	0.220	0.220	0.193	0.507
	RC-1.7-0.05	1.71	0.050	14.6	7.3	0.500	0.208	0.145	0.145	0.242	0.154	0.365
	RC-1.7-0.20	1.71	0.200	18.7	9.8	0.524	0.140	0.196	0.196	0.244	0.168	0.442
	RC-1.7-0.35	1.71	0.350	23.4	12.1	0.517	0.173	0.243	0.221	0.221	0.176	0.391
	RC-1.7-0.50	1.71	0.500	28.9	13.8	0.478	0.197	0.276	0.197	0.197	0.173	0.348
Arakawa <i>et al.</i> [A4]	No. 102	1.50	0.333	20.9	16.7	0.799	0.426	0.596	0.559	0.559	0.441	0.493
Ohue <i>et al.</i> [O1]	2D16RS	2.00	0.143	19.0	14.5	0.763	0.349	0.488	0.488	0.713	0.569	0.725
	4D13RS	2.00	0.153	19.3	15.2	0.788	0.389	0.544	0.544	0.795	0.634	0.760
Ono <i>et al.</i> [O2]	CA025C	1.50	0.257	18.7	14.4	0.770	0.394	0.552	0.552	0.604	0.443	0.591
Umehara <i>et al.</i> [U1]	CUW	1.96	0.162	19.3	16.2	0.839	0.374	0.524	0.524	0.724	0.473	0.810
Bett <i>et al.</i> [B1]	No. 1-1	1.50	0.104	14.7	11.2	0.762	0.16	0.224	0.224	0.368	0.257	0.560
Pujol <i>et al.</i> [P6]	No. 10-2-3N	2.25	0.085	18.8	17.9	0.952	0.511	0.358	0.358	0.597	0.359	0.895
	No. 10-2-3S	2.25	0.085	18.8	19.6	1.043	0.560	0.392	0.392	0.653	0.653	0.980

	No. 10-3-1.5N	2.25	0.089	18.9	18.6	0.984	0.531	0.372	0.372	0.620	0.620	0.930
	No. 10-3-1.5S	2.25	0.089	18.9	21.2	1.122	0.606	0.424	0.424	0.707	0.707	1.060
	No. 10-3-3N	2.25	0.096	19.1	19.4	1.016	0.554	0.388	0.388	0.647	0.647	0.970
	No. 10-3-3S	2.25	0.096	19.1	20.4	1.068	0.583	0.408	0.408	0.680	0.680	1.020
	No. 10-3-2.25N	2.25	0.105	19.4	21.4	1.103	0.306	0.428	0.428	0.713	0.713	1.070
	No. 10-3-2.25S	2.25	0.105	19.4	20.6	1.062	0.294	0.412	0.412	0.687	0.687	1.030
	No. 20-3-3N	2.25	0.158	21.2	22.7	1.071	0.324	0.454	0.454	0.634	0.634	1.087
	No. 20-3-3S	2.25	0.158	21.2	25.0	1.179	0.357	0.500	0.500	0.698	0.698	1.197
	No. 10-2-2.25N	2.25	0.082	18.7	18.8	1.005	0.537	0.376	0.376	0.627	0.627	0.940
	No. 10-2-2.25S	2.25	0.082	18.7	20.2	1.080	0.577	0.404	0.404	0.673	0.673	1.010
	No. 10-1-2.25N	2.25	0.078	18.6	18.8	1.011	0.537	0.376	0.376	0.627	0.627	0.940
	No. 10-1-2.25S	2.25	0.078	18.6	19.5	1.048	0.557	0.390	0.390	0.650	0.650	0.975
Priestley <i>et al.</i> [P3]	R1A	2.00	0.054	16.4	20.0	1.220	0.571	0.400	0.400	0.667	0.667	0.928
	R3A	2.00	0.059	16.6	20.3	1.223	0.580	0.406	0.406	0.677	0.677	0.922
	R5A	1.50	0.063	13.7	17.1	1.248	0.489	0.342	0.342	0.570	0.570	0.855
Esaki <i>et al.</i> [E6]	H-2-1/5	2.00	0.200	20.8	23.6	1.135	0.337	0.472	0.472	0.590	0.590	1.116
	HT-2-1/5	2.00	0.200	20.8	19.6	0.942	0.280	0.392	0.392	0.490	0.490	0.922
	H-2-1/3	2.00	0.334	25.5	28.1	1.102	0.401	0.562	0.526	0.526	0.526	0.982
	HT-2-1/3	2.00	0.333	25.4	26.1	1.028	0.373	0.522	0.489	0.489	0.489	0.914
Mean							0.897	0.382	0.388	0.377	0.534	0.804
Coefficient of Variation							0.235	0.147	0.110	0.113	0.18	0.241

6.8 Conclusion

This chapter presents the analytical method to estimate the initial stiffness of RC columns. A comprehensive parametric study is carried out based on the proposed method to investigate the influences of several critical parameters. A simple equation to estimate the initial stiffness of RC columns is also proposed in this chapter. The following provides specific findings of the chapter:

1. Comparisons made between the analytical results and the experimental results of the ten specimens tested in this research have showed relatively good agreement with each other. This shows the applicability and accuracy of the proposed method to estimate the initial stiffness of RC columns.
2. The parametric study based on the proposed method shows that the stiffness ratio (k) increases with an increase in aspect ratios (R_a) and axial load ratio (R_n). The transverse and longitudinal reinforcement ratios, yield strength of longitudinal bars and concrete compressive strength insignificantly influenced the stiffness ratio.
3. It was found that the average ratio of the experimental to predicted stiffness ratio by the proposed equation is 0.897, showing a good correlation between the proposed equation and the experimental data. The proposed equation may be suitable as an assessment tool to calculate the stiffness ratio of RC columns within the range of the parametric study.

CHAPTER 7

DISPLACEMENT AT AXIAL FAILURE OF RC COLUMNS WITH LIGHT TRANSVERSE REINFORCEMENT

7.1 Introduction

Most tests of reinforced concrete (RC) columns subjected to seismic loading have been terminated shortly after a loss of lateral load resistance. Only few experimental research studies have been carried out to the point of axial failure. These studies had been reviewed in **Chapter 2**. In addition, an experimental program consisting of ten RC columns with light transverse reinforcement tested to the point of axial failure was conducted in this research. The test results obtained from the current and previous research studies had formed a useful database to assess the displacements at axial failure of RC columns with light transverse reinforcement.

This chapter is devoted to develop a simple model to estimate the ultimate displacements or displacements at axial failure of RC columns with light transverse reinforcement subjected to seismic loading. The database presented in **Chapter 2** is used to calibrate the developed model. The applicability and accuracy of the proposed model are then verified with the experimental results presented in **Chapter 4**. Comparisons between the proposed and Elwood *et al.* [E5]'s equations are also presented within this chapter.

7.2 Observed Seismic Performance of RC Columns with Light Transverse Reinforcement

The available literature on the post-earthquake investigations [E2, E3, E4, M1, S3] highlight various types of failures of RC columns with light transverse reinforcement. One of the most critical and typical mode of failure in these types of

columns is often caused by the formation of a steep shear crack as illustrated in **Figure 7.1** and **7.2**. The sliding between the surfaces of this shear crack results in an excessive shear deformation of the columns. This leads to a sudden loss of axial capacity. This type of failure was also observed in the current experimental investigation as shown in **Figure 7.3 (a)**. Any axial load supported by such damaged columns must be transferred through the obvious shear failure plane as shown in **Figure 7.3**.

Figure 7.1 Damaged Column during 1999 Kocaeli Earthquake
(reprinted from Elwood *et al.* 2005 [E5])

Figure 7.2 Damaged Columns during 1994 Northridge, Calif. Earthquake
(reprinted from Sezen *et al.* 2004 [S2])

7.3 Proposed Model

7.3.1 Basic Assumptions

The following summarises the basic assumptions employed in the proposed model:

1. The applied axial load at the point of axial failure is transferred through the shear failure plane as illustrated in **Figure 7.3**.
2. The angle of the shear failure plane of 60 degrees as defined by Priestley *et al.* [P3] is adopted.
3. The shear demand in columns is considered to be negligible and therefore ignored at the point of axial failure [L2, S1].
4. When the shear strength starts to degrade corresponding to a displacement ductility of 1 and 2 for bidirectional and unidirectional lateral loading, respectively [P3], the additional deformation of columns is assumed to be only contributed from the sliding of cracking surfaces as shown in **Figure 7.3**.

7.3.2 Derivation of the Proposed Model

At the point of axial failure as shown in **Figure 7.3**, the external and internal works developed by the column are given as:

$$W_{ext} = P\Delta_{av}^* \quad (7.1)$$

$$W_{int} = W_c + W_{sv} + W_{sl} \quad (7.2)$$

where W_{ext} and W_{int} are the external and internal work done respectively; P is the

applied column axial load; W_c , W_{sv} and W_{sl} are the internal work done by the concrete, transverse reinforcement and longitudinal reinforcement respectively; Δ_a^* and Δ_{av}^* are the horizontal and vertical displacements due to the sliding between cracking surfaces at the point of axial failure.

Figure 7.3 Assumed Failure Plane at the Point of Axial Failure

The internal work done by the longitudinal reinforcement, transverse reinforcement and concrete are calculated as:

$$W_{sl} = (\rho_l b h f_{sl}) \times \Delta_{av}^* \quad (7.3)$$

$$W_{sv} = \left(\frac{d \tan \theta}{s} A_{st} f_{yt} \right) \times \Delta_a^* = \left(\frac{d \tan \theta}{s} A_{st} f_{yt} \right) \times (\Delta_{av}^* \cot \theta) = \frac{d f_{yt} A_{st} \Delta_{av}^*}{s} \quad (7.4)$$

$$W_c = V_c \times \Delta_a^* = V_c \times (\Delta_{av}^* \cot \theta) \quad (7.5)$$

where ρ_l is the longitudinal reinforcement ratio; b and h are the width and depth of columns respectively; f_{sl} is the axial strength of longitudinal reinforcement at axial failure; f_{yt} is the yield strength of transverse reinforcement; d is the distance from the extreme compression fiber to centroid of tension reinforcement; s is the spacing of transverse reinforcement; A_{st} is the total transverse reinforcement area within spacing s ; θ is the angle of shear crack and V_c is the shear force carried by concrete.

By substituting Equations 7.3, 7.4 and 7.5 into 7.2 and equating Equations 7.1 and 7.2, it gives:

$$P\Delta_{av}^* = \rho_l b h f_{sl} \Delta_{av}^* + \frac{df_{yt} A_{st} \Delta_{av}^*}{s} + V_c \Delta_{av}^* \cot \theta \quad (7.6a)$$

or

$$P = \rho_l b h f_{sl} + \frac{df_{yt} A_{st}}{s} + V_c \cot \theta \quad (7.6b)$$

Equation 7.6 can be rewritten as:

$$P = P_{sl} + P_{st} + P_c \quad (7.7)$$

in which:

$$P_{sl} = \rho_l b h f_{sl} \quad (7.8)$$

$$P_{st} = \frac{df_{yt} A_{st}}{s} \quad (7.9)$$

$$P_c = V_c \cot \theta \quad (7.10)$$

where P_{sl} , P_{st} and P_c are the axial strength contributed by longitudinal

reinforcement, transverse reinforcement and concrete at the point of axial failure respectively.

The axial strength of longitudinal reinforcing bars at axial failure can be calculated as follows:

$$f_{sl} = \frac{(P - P_{st} - P_c)}{\rho_l b h} \quad (7.11)$$

Hence the ratio of the axial strength of longitudinal reinforcing bars at axial failure to the yield strength of longitudinal reinforcement (η_{sl}) is given by:

$$\eta_{sl} = \frac{(P - P_{st} - P_c)}{\rho_l b h f_{yl}} \quad (7.12)$$

Figure 7.4 Definition of parameter k (reprinted from Priestley *et al.* 1994 [P3])

Based on Priestley's model [P3], the shear force carried by concrete is calculated as follows:

$$V_c = k \sqrt{f'_c} (0.8 A_g) \quad (7.13)$$

in which A_g is the cross sectional area and the parameter k depends on the displacement ductility demand as defined in **Figure 7.4**.

The horizontal displacement of columns at the point of axial failure is calculated as:

$$\Delta_a = \Delta_a^* + 2\Delta_y \text{ for Unidirectional Lateral Loading} \quad (7.14a)$$

or

$$\Delta_a = \Delta_a^* + \Delta_y \text{ for Bidirectional Lateral Loading} \quad (7.14b)$$

Hence the horizontal displacement due to the sliding between cracking surfaces at the point of axial failure is given as:

$$\Delta_a^* = \Delta_a - 2\Delta_y \text{ for Unidirectional Lateral Loading} \quad (7.15a)$$

or

$$\Delta_a^* = \Delta_a - \Delta_y \text{ for Bidirectional Lateral Loading} \quad (7.15b)$$

where Δ_y is the yield displacement defined as follows: a secant was drawn to intersect the lateral-displacement relation at the yield force. This line was extended to the intersection with a horizontal line corresponding to the flexural strength, and then projected onto the horizontal axis to obtain the yield displacement.

The damaged length (L_d) as shown in **Figure 7.3** is given by:

$$L_d = h \tan \theta \quad (7.16)$$

The ratio of the horizontal displacement due to the sliding between cracking surfaces at axial failure to the damaged length (δ_a^*) is given as follows:

$$\delta_a^* = \frac{\Delta_a - 2\Delta_y}{h \tan \theta} \times 100\% \quad (7.17)$$

7.3.3 Calibration of the Proposed Model

A database consisting of 48 RC columns tested to the point of axial failure was been constructed in **Chapter 2**. Details of these RC columns are shown in **Table 2.1**. These columns encompass a wide range of cross sectional sizes, material properties, and column axial loads. These columns were subjected to a combination of an axial load and cyclic loadings to simulate earthquake actions.

Table 7.1 shows the experimental ratios of $(\eta_{sl})_{exp}$ and $(\delta_a^*)_{exp}$ for each of the test column in the database, which are calculated based on Equations 7.12 and 7.17 respectively. **Figure 7.5** plots $(\eta_{sl})_{exp}$ versus $(\delta_a^*)_{exp}$ for all test columns in the database. Based on the results from the database, an empirical equation was then developed to relate the ratio of the axial strength of the longitudinal reinforcing bars to the yield strength of the longitudinal reinforcing bars (η_{sl}) to the ratio of the horizontal displacement due to the sliding between cracking surfaces to the damaged length (δ_a^*) as follows:

$$\eta_{sl} = \frac{1}{0.2046 \times \delta_a^* + 1} \quad (7.18)$$

Figure 7.5 Relationship between η_{sl} and δ_a^*

Table 7.1 Calculated Values of η_{sl} and δ_a^* for RC Columns in the Database

	Specimen	$(\Delta_a^*)_{exp}$ (mm)	$(\delta_a^*)_{exp}$	$(\eta_{sl})_{exp}$	$(\eta_{sl})_{pro}$	$\frac{(\eta_{sl})_{exp}}{(\eta_{sl})_{pro}}$
Yoshimura <i>et al.</i> [Y1]	FS0	46.7	8.99	0.32	0.352	0.909
	FS0	42.5	8.18	0.32	0.374	0.856
	FS0	27.9	5.36	0.32	0.477	0.671
	S1	41.3	5.96	0.197	0.45	0.438
	S2A	47.7	6.88	0.197	0.415	0.475
	S2A	35.1	5.06	0.197	0.491	0.401
Lynn <i>et al.</i> [L2]	3CLH18	22.4	2.83	0.167	0.633	0.264
	2CLH18	56.8	7.17	0.293	0.405	0.723
	3SLH18	52.8	6.66	0.185	0.423	0.437
	2SLH18	72.1	9.11	0.293	0.349	0.84
	2CMH18	0.0	0.00	0.961	1.000	0.961
	3CMH18	21.5	2.72	0.623	0.643	0.969
	3CMD12	21.5	2.72	0.587	0.643	0.913
	3SMD12	21.5	2.72	0.589	0.643	0.916
Sezen <i>et al.</i> [S1]	2CLD12	108.0	13.7	0.196	0.263	0.745
	2CHD12	6.8	0.86	0.991	0.851	1.165
	2CVD12	44.4	5.61	0.535	0.465	1.151
	2CLD12M	123.0	15.6	0.196	0.239	0.82
Nakumura <i>et al.</i> [N1]	N18M	84.9	16.3	0.386	0.23	1.678
	N18C	178	34.2	0.386	0.125	3.088
	N18C	33.4	6.42	0.874	0.432	2.023
	N18C	18.1	3.48	0.874	0.584	1.497
Yoshimura <i>et al.</i> [Y2]	2M	62.2	12.0	0.37	0.29	1.276
	2C	41.8	8.05	0.37	0.378	0.979
	3M	27.9	5.37	0.607	0.477	1.273
	3C	26.1	5.02	0.607	0.493	1.231
	2M13	20.4	3.93	0.634	0.555	1.142
	2C13	13.8	2.66	0.634	0.648	0.978

Yoshimura <i>et al.</i> [Y3]	No.1	146.0	28.0	0.488	0.148	3.297
	No.2	51.3	9.87	0.507	0.331	1.532
	No.3	9.4	1.81	0.501	0.73	0.686
	No.4	9.7	1.87	0.758	0.723	1.048
	No.5	9.3	1.79	0.898	0.732	1.227
	No.6	50.8	9.77	0.726	0.333	2.18
	No.7	12.3	2.37	0.755	0.674	1.12
Ousalem <i>et al.</i> [O3]	C1	2.3	0.44	0.541	0.917	0.59
	C4	29.5	5.68	0.507	0.462	1.097
	C8	7.15	1.38	0.724	0.78	0.928
	C12	66.6	12.8	0.425	0.276	1.54
	D1	19.0	3.66	0.563	0.572	0.984
	D16	18.7	3.60	0.564	0.576	0.979
	D11	8.1	1.56	0.501	0.758	0.661
	D12	8.6	1.66	0.504	0.746	0.676
	D13	22.5	4.33	0.423	0.53	0.798
	D14	81.5	15.7	0.423	0.238	1.777
	D15	152.0	29.2	0.300	0.143	2.098
	D5	38.8	7.46	0.385	0.396	0.972
Tran <i>et al.</i> [T1]	SC01	6.6	1.089	1.035	0.818	1.265
Mean						1.131
Coefficient of Variation						0.609

7.4 Verification of the Proposed Model

The experimental results presented in **Chapter 4** are used to validate the proposed model with respect to the displacement at axial failure.

The displacements at axial failure of the test specimens are calculated based on Equations 7.9, 7.10, 7.12, 7.13, 7.17 and 7.18. It was found that the average ratio of the experimental to predicted displacement at axial failure by the proposed equation was 1.076 as shown in **Figure 7.6** and **Table 7.2**, showing a good correlation between the proposed equation and experimental data.

Figure 7.6 Comparisons between Experimental and Analytical Ultimate Displacements of Various Equations

The displacements at axial failure obtained from Elwood *et al.*'s model [E5] are also tabulated in **Table 7.2** for comparison with the proposed method. Elwood *et al.*' [E5] proposed the following equation for the drift ratio at axial failure based on a shear friction model with the free body diagram as shown in **Figure 7.7**:

$$\left(\frac{\Delta}{L}\right)_a = \frac{4}{100} \frac{1 + (\tan \theta)^2}{\tan \theta + P \left(\frac{s}{A_{st} f_{yt} d_c \tan \theta} \right)} \quad (7.19)$$

where d_c is the depth of core (centerline to centerline of ties).

Table 7.2 Experimental Verification of the Proposed Model

Specimen	$(\Delta_a)_{\text{exp}}$ (mm)	$(\Delta_a)_{\text{pro}}$ (mm)	$(\Delta_a)_{\text{Elwood}}$ (mm)	$\frac{(\Delta_a)_{\text{exp}}}{(\Delta_a)_{\text{pro}}}$	$\frac{(\Delta_a)_{\text{exp}}}{(\Delta_a)_{\text{Elwood}}}$
SC-2.4-0.20	48.0	53.5	51.1	0.897	0.939
SC-2.4-0.50	28.6	26.0	23.0	1.100	1.243
SC-1.7-0.05	135.5	205.1	69.9	0.601	1.938
SC-1.7-0.20	21.8	28.8	31.5	0.757	0.692
SC-1.7-0.35	18.7	14.4	21.5	1.299	0.87
SC-1.7-0.50	17.0	14.0	14.8	1.214	1.149
RC-1.7-0.05	192.5	315.2	116.1	0.611	1.658
RC-1.7-0.20	48.8	47.5	45.3	1.027	1.078
RC-1.7-0.35	34.4	23.8	28.3	1.445	1.215
RC-1.7-0.50	30.6	16.9	21.5	1.811	1.426
Mean				1.076	1.220
Coefficient of Variation				0.384	0.373

The mean ratios of the experimental to the predicted displacement at axial failure and its coefficient of variation are 1.076 and 0.384 for the proposed model and 1.220 and 0.373 for Elwood *et al.*'s equation [E5] respectively. Comparing the two models with experimental data indicates that the proposed model produced a better mean ratio of the experimental to the predicted displacement at axial failure than Elwood *et al.*'s equation [E5]. It is to be noted that Elwood *et al.*'s equation [E5] was developed based on the test data of columns experiencing flexure-shear failures. Whereas, a majority of the test columns in the current experimental program experienced pure shear failures. This difference in the failure mode has resulted in

the inaccuracy of Elwood *et al.*'s equation [E5] when applying for the test columns in the current experimental program.

Figure 7.7 Free Body Diagram of Column after Shear Failure
(reprinted from Elwood *et al.* 2005 [E5])

7.5 Applicability of the Proposed Model for Backbone Curves of RC Columns with Light Transverse Reinforcement

The test results presented in **Chapters 4** and **5** showed that the backbone curves based on both FEMA 356 [F1] and ASCE 41 [E8] guidelines showed a good prediction of the shear strength of the test specimens. However, the column initial stiffness and ultimate displacements (displacements at axial failure) were over-estimated and under-estimated by both FEMA 356 [F1] and ASCE 41 [E8] guidelines, respectively. Therefore, further works on the initial stiffness and ultimate displacement of RC columns are needed to accurately capture the behavior of RC columns tested to the point of axial failure.

Research works on the initial stiffness and the ultimate displacements of RC columns had been done in **Chapter 6** and the beginning part of this chapter, respectively. It was found that the developed stiffness and ultimate displacement

models in **Chapter 6** and the beginning part of this chapter produced better results than the existing models, respectively. In this part of the chapter, the FEMA 356's backbone curve [F1] is modified based on the analytical results obtained from **Chapters 6** and **7**. The modified FEMA 356's backbone curve is shown in **Figure 7.8**.

The point B in **Figure 7.8** is defined based on K_i and V_p . Where the initial stiffness K_i is calculated based on the developed model in **Chapter 6**; V_p is equal to the minimum value of the theoretical yield force V_y and the nominal shear strength based on FEMA 356's model [F1] V_n .

The point C in **Figure 7.8** is defined based on a and V_m . Where a is defined similarly to FEMA 356's model [F1]; V_m is equal to the minimum value of the theoretical flexural strength V_u and the nominal shear strength based on FEMA 356's model [F1] V_n .

The point E in **Figure 7.8** is defined based on c and Δ_a . Where c is defined similarly to FEMA 356's model [F1]; the ultimate displacement Δ_a is calculated based on Equations 7.9, 7.10, 7.12, 7.13, 7.17 and 7.18.

Figure 7.8 Modified FEMA 356 [F1]'s backbone for RC Columns with Light Transverse Reinforcement

Figure 7.9 shows the comparison of available models with the test results obtained from the current experimental investigation. The backbone curves obtained from Elwood *et al.*'s model [E9] are also illustrated in **Figure 7.9** for comparison with the proposed method. Elwood *et al.*'s [E9] backbone model as shown in **Figure 7.10** is defined based on the yield displacement Δ_y , displacement at shear failure Δ_s , displacement at axial failure Δ_a and shear strength at zero displacement ductility V_n .

Where the yield displacement Δ_y , displacement at axial failure Δ_a and shear strength at zero displacement ductility V_n are calculated based on Equations 6.1, 7.19 and 3.2, respectively. The displacement at shear failure Δ_s is given as:

$$\Delta_s = \frac{3}{100} + 4\rho_v - \frac{1}{40} \frac{v}{\sqrt{f'_c}} - \frac{1}{40} \frac{P}{A_g f'_c} \geq \frac{1}{100} \text{ (in MPa)} \quad (7.20)$$

where ρ_v is the transverse steel ratio; v is the nominal shear stress; f'_c is the concrete compressive strength, A_g is the gross cross sectional area and P is the applied column axial load.

(a)

(b)

(c)

(d)

(e)

(f)

(g)

(h)

Figure 7.9 Comparison between Experimental Backbone Curves and Proposed Model

Figure 7.10 Elwood *et al.*' [E9] Backbone Model

Comparison of available models with the test results obtained from the current experimental investigation as illustrated in **Figure 7.9** indicated that the proposed model and Elwood *et al.*'s model [E9] provided a better prediction of the behavior of the test specimens than the FEMA 356 [F1] and ASCE 41 [E8]'s model. The initial stiffness and the ultimate displacement were fairly captured by the proposed model and Elwood *et al.*'s model [E9]. The proposed method and Elwood *et al.*'s model [E9] may be suitable as an assessment tool to model the backbone curves of RC columns with light transverse reinforcement.

7.6 Conclusion

An analytical model is developed in this chapter to estimate the displacement at the point of axial failure of RC columns with light transverse reinforcement. The following provides specific findings of the chapter:

1. An empirical equation is developed to relate the ratio of the axial strength of longitudinal reinforcing bars to the yield strength of longitudinal reinforcing bars (η_{sl}) to the ratio of the horizontal displacement due to the sliding between cracking surfaces to the damaged length (δ_a^*) by using the test results of RC columns in the database presented in **Chapter 2**.

2. The developed model is validated by using the test results presented in **Chapter 4**. It was found that the average ratio of the experimental to the predicted displacement at axial failure by the proposed method was 1.076, showing a good correlation between the proposed equation and experimental data. The proposed equation may be suitable as an assessment tool to calculate the displacement at axial failure of RC columns with light transverse reinforcement.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Introduction

The seismic performance of reinforced concrete columns with light transverse reinforcement was investigated through experimental and analytical means in this study.

The study presented within this report consists of three main parts. In the first part of the report, an experimental program carried out on ten reinforced concrete columns with light transverse reinforcement to the point of axial failure was presented. The variables in the test specimens include column axial loads, aspect ratios, and cross sectional shapes. The test results were compared with existing seismic assessment models.

In the second part of the report, an analytical approach, coupling flexure and shear deformations, was proposed to evaluate the initial stiffness of reinforced concrete columns subjected to seismic loading. A comprehensive parametric study was carried out based on the proposed approach to investigate the influences of several critical parameters and a simple equation was proposed to estimate the initial stiffness of reinforced concrete columns.

Finally, an empirical model was developed to estimate the displacement at axial failure of reinforced concrete columns with light transverse reinforcement subjected seismic loading. The proposed model was calibrated with the available database of reinforced concrete columns tested to the point of axial failure.

Conclusions drawn from the experimental and analytical results will be presented in the following sections. Recommendations on future works will also be presented.

8.2 Experimental Investigations

The conclusions drawn from the experimental investigations of ten reinforced concrete columns with light transverse reinforcement are as follows:

1. There were two distinctive modes of shear failure observed in the test specimens. In the first mode, the failure was controlled by a steep shear crack, whereas shear and bond-splitting cracks controlled the failure in the second mode.
2. There were two modes of axial failure observed in the test specimens. In the first mode, the steep shear crack developed on the column from the previous stages became wider. This led to sliding between the cracking surfaces as well as the buckling of longitudinal reinforcing bars and fracturing of transverse reinforcing bars along this shear crack. In the second mode, crushing of concrete together as well as the buckling of longitudinal reinforcing bars and fracturing of transverse developed across a damaged zone.
3. The column axial load was found to have a detrimental effect on the drift ratio at axial failure and maximum energy dissipation capacity of test specimens. However, the shear strength and initial stiffness increased with an increase in column axial load.
4. The drift ratio at axial failure and maximum energy dissipation capacity of test specimens dropped with a decrease in aspect ratio. However, the shear strength and initial stiffness increased with a decrease in aspect ratio.
5. Better seismic performances were observed in RC-1.7 Series specimens as compared to SC-2.4 and SC-1.7 Series specimens at the same column axial load.

6. A decrease in aspect ratio and an increase in column axial load have led to the predominance of shear over flexural deformations. This is a clear sign of the transition from a flexural to shear mode of failure, which subsequently leads to a reduction in the drift ratio at axial failure in the test specimens.
7. The test results showed that both FEMA 356 [F1] and ASCE 41 [E8] guidelines provided a good prediction of the shear strength of the test specimens. However, the column initial stiffness and ultimate displacements (displacements at axial failure) were over-estimated and under-estimated by both FEMA 356 [F1] and ASCE 41 [E8] guidelines, respectively.

8.3 Analytical Investigations

8.3.1 Initial Stiffness

The conclusions drawn from the analytical investigation regarding to the initial stiffness of RC columns are as follows:

1. Comparisons made between the analytical results and the experimental results of the ten specimens tested in this research have shown relatively good agreement with each other. This shows the applicability and accuracy of the proposed method to estimate the initial stiffness of RC columns.
2. The parametric study based on the proposed method shows that the stiffness ratio (k) increases with an increase in aspect ratios (R_a) and axial load ratio (R_n). The transverse and longitudinal reinforcement ratios, yield strength of longitudinal bars and concrete compressive strength insignificantly influenced the stiffness ratio.
3. It was found that the average ratio of the experimental to predicted stiffness ratio by the proposed equation is 0.897, showing a good correlation between the proposed equation and the experimental data. The proposed equation

may be suitable as an assessment tool to calculate the stiffness ratio of RC columns within the range of the parametric study.

8.3.2 Displacement at Axial Failure

An analytical model is developed to estimate the displacement at the point of axial failure of RC columns with light transverse reinforcement. The conclusions drawn are as follows:

1. An empirical equation is developed to relate the ratio of the axial strength of longitudinal reinforcing bars to the yield strength of longitudinal reinforcing bars (η_{sl}) to the ratio of the horizontal displacement due to the sliding between cracking surfaces to the damaged length (δ_a^*) by using the test results of RC columns in the available database.
2. The developed model is validated by using the test results in the current experimental investigation. It was found that the average ratio of the experimental to the predicted displacement at axial failure by the proposed method was 1.076, showing a good correlation between the proposed equation and experimental data. The proposed equation may be suitable as an assessment tool to calculate the displacement at axial failure of RC columns with light transverse reinforcement.

8.4 Recommendations for Future Works

It is not possible to provide a complete investigation in this study. Therefore the following experimental and analytical researches are recommended to obtain a better understanding in the seismic behavior of reinforced concrete columns with light transverse reinforcement:

1. The influences of loading direction on the drift ratio at axial failure and initial stiffness are needed to further study.

2. Further experimental and analytical studies are needed to understand the effects of loading history on the drift ratio at axial failure and initial stiffness. The pulse-like type of loading representing the effects of near-fault earthquakes, which are known to impose sudden large displacements without many cycles to surrounding structures, should be paid more attention.
3. The effects of bidirectional loading on the drift ratio at axial failure and initial stiffness should be further studied both experimentally and analytically.
4. In this research, the column depth to width ratio of rectangular specimens was 1.96. The seismic behavior of wall-like columns with a high ratio of column depth to width to the point of axial failure is worth to further research.
5. The influences of different types of cross section on the seismic behavior of RC columns with non-seismic details should be further studied.
6. Tests on combined columns (frames) are needed to obtain the drift ratio at axial failure of frames.
7. Further studies should be carried out to quantify the impact of using different stiffness models on various issues of structural analysis of RC frames subjected to seismic loadings, such as distribution of internal forces and the fundamental period of the structures.

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