Seismic assessment of existing reinforced concrete columns

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**SEISMIC ASSESSMENT OF
EXISTING REINFORCED CONCRETE COLUMNS**

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2014

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EXISTING REINFORCED CONCRETE COLUMNS**

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SCHOOL OF CIVIL & ENVIROMENTAL ENGINEERING

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SUMMARY

Existing Reinforced Concrete (RC) columns, commonly designed and built with light transverse reinforcement, were prevalently used in buildings worldwide before significant advancements in building code provisions were instituted in the mid-1970s. Post-earthquake investigations have shown that these columns are vulnerable to earthquake-induced collapse, posing a threat to public safety in future earthquakes. In this context, seismic assessment of existing RC columns plays a key role and needs further development.

In this research, two series of experimental program consisting of sixteen column specimens were investigated to study the seismic behavior and failure mechanism of existing RC columns. The first test series was conducted on nine RC columns having light transverse reinforcement and plain longitudinal reinforcement. The seismic performance of the test specimens was assessed in terms of crack patterns, hysteretic response, initial stiffness, shear strength, drift capacity and energy dissipation capacity. The test results were discussed and compared with available experimental data of similar columns reinforced with deformed rebars. The comparisons highlighted that apart from lower shear capacities; columns with plain rebars had relatively large initial stiffness and drift capacities, even higher than columns with deformed longitudinal rebars. Further comparisons with the existing model for evaluation of existing structures revealed that the ASCE 41 provision tends to over-estimate initial stiffness, shear strength but substantially underestimates drift capacities of columns with plain rebars. A reduction factor therefore was proposed to improve the accuracy of ASCE 41 when employed for shear strength evaluation of RC columns with plain longitudinal reinforcement.

An analytical model was developed to study the unique failure mechanism observed from the first test series. A new load-transferring path was introduced which is not

only capable of explaining the failure mechanism but also allows for the development of a shear strength model for the column specimens. To enhance the applicability of the model, it was further corroborated with 89 short columns screened from existing literature. Good agreement between analytical and experimental results was achieved.

To further understand the seismic response of existing RC columns, a second test series was conducted in this study with the focus on the effects of seismic loading directions. Seven RC columns with light transverse reinforcement and having deformed rebars were tested under non-principal loading directions. The obtained test results were analyzed, discussed and compared with similar columns previously tested under principal directions. The direction of seismic loads was found to have a significant effect on the drift capacities and failure modes of both rectangular and square columns. The extent of change in shear strength resulting from the different angles of seismic loading directions could be predicted. A Finite Element Model was developed to supplement the results of the second test series. Based on both experimental and numerical results, a simple upper limit for shear strength contributed by concrete was proposed.

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CHAPTER I: INTRODUCTION

1.1 Problem Statement

The world experienced many devastating and deadly earthquakes over the past decades. The Northridge earthquake in California, America (1994); The Hyogo-ken Nanbu earthquake in Kobe, Japan (1995); Ji-Ji Earthquake in Taiwan (1999) and Christchurch Earthquake in New Zealand (2011) are but a few examples. Observations found that a larger number of existing reinforced concrete (RC) buildings (or older buildings) were collapsed while modern buildings generally performed well. This is due to the fact that the older buildings were designed to low levels of seismic loads according to older design codes which resulted in low seismic capacity of columns - the key structural components. Therefore, such columns are inherently vulnerable to earthquakes.

To increase the seismic capacity of newly constructed buildings, significant improvements were added into the 1976 Uniform Building Code,⁽¹⁾ in the United State. Similar changes were also made in the enforcement of Building Standard Law 1981⁽²⁾ in Japan and the building code NZS 4203:1976⁽³⁾ in New Zealand Standard. These changes generally resulted in the design of RC columns with greater ductility than those designed in accordance with older building codes. In this context, structural safety of existing RC buildings is one of the key issues of modern earthquake and structural engineering. Addressing this concern, structural strengthening seems to be the most effective method to improve the seismic performance of existing buildings those

commonly are historical or other important buildings. To accomplish this task, seismic assessment of existing RC columns draws an urgent need for international research attention.

Over the last twenty years, a number of experimental and analytical studies have been conducted to gain better understanding on the seismic behavior of existing RC columns. However, provided the current stage of knowledge, structural engineers still face major challenges in determining seismic capacity and in explaining failure mechanism of such columns. This reason strongly prompts the research presented in this dissertation.

1.2 Characteristics of Existing RC Columns

Surveys conducted on existing RC buildings (Moehle and Mahin 1991;⁽⁴⁾ Rutherford and Chekene 1992 and 1995;⁽⁵⁾ Lynn et al. 1996;⁽⁶⁾ Taciroglua et al. 2008;⁽⁷⁾ Günay, M. S. and K. M. Mosalam⁽⁸⁾) found that columns in these structures typically had widely spaced and poor arrangement of transverse reinforcement. This configuration results in inadequate confinement of the concrete in the column core for ductility demands related to seismic action.

Existing concrete columns reinforced with plain reinforcing bars were found in many buildings which were generally built before the 1970s,^{(9),(10)} mostly in Europe, Asia and Oceania. Due to the use of plain rebars, bonding between reinforcing bars and surrounding concrete is limited, and therefore internal forces cannot be transferred by mechanical interlock. This phenomenon might significantly affect the behavior of the columns.

Earthquake reconnaissance and experimental studies have shown that existing columns with aforementioned configurations are vulnerable to shear failure under the effect of earthquake loading (example shown in **Figure 1.1**). As shear failure progresses, axial load capacity of the member might be reduced and inertial gravity load are transferred

to adjacent members, leading to total building collapse (as depicted in **Figure 1.2** and **Figure 1.3**).

Figure 1.1 Shear failure in existing RC column due to insufficient shear reinforcement⁽¹¹⁾

Figure 1.2 Damage to existing RC buildings due to 1994 Northridge Earthquake
(Photos procured from USGS archives⁽¹²⁾)

Collapse of a residential building in
the 1995 Kobe (Japan) earthquake

Poor performance of an Existing
RC column

Figure 1.3 Axial failure due to earthquake

1.3 Objectives and Scope

The overall objective of this research is to evaluate and model the seismic behavior of existing RC columns. The emphasis is given to the effects of four parameters: the shear span to depth ratio, cross sectional shape, level of applied axial force, and seismic loading direction. The investigated columns are assessed in terms of initial stiffness, shear strength, drift capacity, energy dissipation capacity and failure modes. To achieve such a broad objective, the scope of the research is as follows:

1. To experimentally study the seismic behavior of existing concrete columns reinforced with plain longitudinal reinforcement and light transverse reinforcement.
2. To experimentally study the effects of seismic loading directions on the performance of existing RC columns with deformed rebars and light transverse reinforcement.
3. To develop an analytical model that is capable of capturing the failure mechanism and of estimating shear strength for existing RC columns.

1.4 Dissertation Outline

This dissertation consists of seven chapters and is outlined as illustrated in **Figure 1.4**. The introduction presented in Chapter 1 is followed by a review of relevant research presented in the next chapter. Chapter 2 provides an overview on recent experimental studies on (i) existing RC columns having light transverse reinforcement and (ii) RC columns with plain longitudinal reinforcement. A review on current shear strength models is also given in the later part of this chapter.

Chapter 3 describes an experimental program including the design of two test series, the test set-up and instrumentation. The first test series will be conducted on the seismic performance of existing RC columns having plain reinforcing bars with three parameters: the shear span to depth ratio, cross sectional shape and level of applied axial force. The second test series aim to investigate the effects of seismic loading direction on the response of existing RC columns.

Chapter 4 presents and discusses the results of the first test series. The performance of specimens is discussed in terms of crack patterns, hysteretic response, initial stiffness, shear strength, drift capacity and energy dissipation capacity. The test results are compared with previous experimental data of similar concrete columns but reinforced with deformed rebars. Further comparisons are made with the predictions from current code for evaluation of existing structures - ASCE 41.

In Chapter 5, a new load-transferring path is proposed to explain the observed failure mechanism of the test specimens in Series 1. Based on the proposed mechanism, an analytical model to estimate the shear strength of the test specimens is developed. The model is then further developed to enhance its applicability and verified with the test results of 89 columns.

Chapter 6 discusses the experimental results of the second test series. A three dimensional finite element model is developed to supplement the experimental results. A simple upper limit for shear strength contributed by concrete is proposed.

Summary, conclusions and recommendations for future work are discussed in Chapter 7.

Figure 1.4 Organization of the dissertation

CHAPTER II: LITERATURE REVIEW

The preceding chapter has dawned an urgent need to conduct an investigation on the seismic response of existing RC columns. In this chapter, a literature review on the related topics is presented in three sections. In Section 2.1 recent experimental studies conducted on existing RC columns with deformed rebars and light transverse reinforcement are reviewed. Experimental studies on RC columns having plain longitudinal reinforcement are summarized in Section 2.2. Section 2.3 reviews available models for predicting shear strength of RC columns. Shortcomings in previous studies are identified and summarized in the last section.

2.1 Previous Tests on Existing RC Columns with Deformed Rebars and Light Transverse Reinforcement

2.1.1 Lynn et al. (1996)

Lynn et al. (1996)⁽⁶⁾ conducted an experimental program on eight full-scale RC columns having details of existing columns those were typically built before the mid-1970s. The column specimens had light transverse reinforcement and lap-splices at the bottom base as shown in **Figure 2.1**. It was designed with Grade 40 deformed reinforcement and normal weight aggregate concrete with compressive strength in the range of 25.5 to 33.1 MPa. Variables of the tests included the longitudinal reinforcement ratio (total area of rebars divided by cross sectional area) and the transverse reinforcement ratio (total area of transverse reinforcement in one horizontal direction divided by the product of stirrup spacing and column width), details of splices and axial load level. The specimens were loaded with a constant axial load and cyclic lateral deformation until the columns were unable to sustain the vertical load (axial failure). The applied axial load was either $0.12f'_c A_g$ or $0.35f'_c A_g$, where f'_c is the concrete compressive strength and A_g is the column gross cross-sectional area. Results from the tests showed that the predominant response modes were flexure-shear failure, lap-splice failure, and axial failure. It was observed in all specimens splitting cracks near the splices spread into the column body, forming a series of inclined cracks, and leading to shear failure. Axial failure

occurred at or after significant loss of shear resistance. The recorded flexural and shear strengths were consistent with the estimated results calculated using procedures in ACI 318-95. The shear strength of test specimens was not obviously related to displacement demand.

Figure 2.1 Details of specimens tested by Lynn et al. (1996)

2.1.2 Yoshimura (2000, 2003)

Yoshimura and Yamanaka (2000)⁽¹³⁾ conducted an experimental study testing two series of RC columns to observe axial failure. The details of test specimens are shown in **Figure 2.2**. Three loading schemes including monotonic, unidirectional cyclic and bidirectional cyclic loading were investigated in this study. Moderate axial loads of $0.26f_c' A_g$ and $0.20f_c' A_g$ were applied to the first and the second series, respectively. The first series failed in shear after flexural yielding while the second series failed in shear prior to reaching flexural yielding. Results from the tests showed that the lateral and vertical displacements at the ultimate limit state (axial failure) varied depending on the applied loading history. The specimens subjected to bilateral loads obtained the lowest ultimate lateral displacement in both series.

Yoshimura and Nakamura (2003)⁽¹⁴⁾ tested six short RC columns with a shear span to depth ratio of 1.0 as shown in **Figure 2.2**. The effects of applied axial loads, longitudinal reinforcement ratio and loading history on drift ratio at axial failure were studied. It was found that a smaller drift ratio at axial failure was obtained in the specimens with a lower ratio of longitudinal reinforcement.

Figure 2.2 Details of specimens tested by Yoshimura and Yamanaka (2000)

Figure 2.3 Details of specimens tested by Yoshimura and Nakamura (2003)

Yoshimura et al. (2003)⁽¹⁵⁾ carried out a test program on eight half-scale RC columns with details shown in **Figure 2.4**. The shear span to depth ratios of all test specimens were 2.0. 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 the ratios of longitudinal and transverse reinforcement and the applied axial loads. Observation from the tests showed that axial failure of the specimens with shear failure mode (without prior flexural yielding) occurred when the shear resistance degraded nearly to zero. The specimens that failed in flexural-shear failure mode collapsed simultaneously with the onset of shear failure.

Figure 2.4 Details of specimens tested by Yoshimura et al. (2003)

2.1.3 Nakamura and Yoshimura (2002)

Nakamura and Yoshimura⁽¹⁶⁾ conducted four column tests with the configurations of specimens shown in **Figure 2.5**. Test parameters were axial loads and horizontal loading history. Two levels of axial loads, $0.18f_c'A_g$ and $0.27f_c'A_g$, were loaded while horizontal loads were applied either monotonically or cyclically. The tests showed that axial failure was occurred when shear resistance degraded

nearly to zero. 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.

Figure 2.5 Details of specimens tested by Nakamura and Yoshimura (2002)

2.1.4 Sezen and Moehle (2006)

Sezen and Moehle (2006)⁽¹⁷⁾ carried out a test program on four full-scale RC columns with insufficient transverse reinforcement to identify the main factors contributing to shear and axial failure of existing RC columns. The column specimens were designed similarly to the specimens in the test of Lynn et al. (1996) and shown in **Figure 2.6**. The primary variables of the test program were the levels of axial load and the lateral displacement history. Two levels of axial load, $0.12f'_c A_g$ or $0.35f'_c A_g$, were applied. Three specimens were tested under cyclic bending and one specimen was tested under monotonic lateral load. Sezen and Moehle found that response of RC columns with identical properties varied considerably with the

magnitude and history of axial and lateral loads. Specimens with low levels of axial load failed in shear at low displacement ductility but sustained axial force at large displacement. On the other hand, with high magnitude of applied axial load, the specimens experienced brittle shear failure and lost their vertical load carrying capacity immediately after shear failure. The deformation capacity of columns under monotonic loading was found to be higher than that of columns under cyclic loading. Lateral stiffness was strongly affected by slip of longitudinal reinforcement at column-base interfaces.

Figure 2.6 Details of specimens tested by Sezen and Moehle (2006)

2.1.5 Ousalem (2006)

Ousalem (2006)⁽¹⁸⁾ carried out two series of test on scaled RC columns. The test specimens had similar details to the specimens in the study of Nakamura and Yoshimura (2002). The first test series had the same lateral loading with different axial load history; some specimens had constant axial load while the others had varying axial load. Results from the first series showed that higher axial load

resulted in a steeper failure plane with lower lateral displacement capacity. The second series were tested under the same constant axial force while varying lateral loading history. It was found that at a particular level of axial force, lower number of cycles resulted in a slightly steeper failure plane. Few reversal loading type resulted in a larger drift capacity compared with many reversal one for high stirrup ratio. Columns with low ratio of transverse reinforcement were not influenced by the loading history.

2.1.6 Tran (2010)

Tran (2010)⁽¹⁹⁾ carried out an experimental program at Nanyang Technological University, Singapore, consisting of four rectangular and six square RC columns with light transverse reinforcement. Details of test specimens are shown in the **Table 2.1** and **Figure 2.7**. The columns were tested under a combination of constant axial load and quasi-static cyclic load to examine seismic behavior and axial failure mechanism. The variables in the test specimens were axial loads, shear span to depth ratio and cross sectional shapes.

Based on the test results, Tran found that there were two distinctive modes of shear failure, leading to two types of axial failure. 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. Vertical loads were found to have a detrimental effect on the drift ratio at axial failure and energy dissipation capacity of the specimens. However, the shear strength and initial stiffness increased with an increase in column axial load. A decrease in shear span to depth ratio and an increment in column axial load led to the predominance of shear over flexural deformations.

Table 2.1 Summary of specimens tested by Tran (2010)

Specimen	Longitudinal Reinforcement	Transverse Reinforcement	f'_c (MPa)	$b \times h$ (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						0.05
SC-1.7-0.20					1200	0.20
SC-1.7-0.35					1700	0.35
SC-1.7-0.50						0.50
RC-1.7-0.05						0.05
RC-1.7-0.20		2-R6 @ 125 $\rho_v = 0.18\%$		250×490	1700	0.20
RC-1.7-0.35						0.35
RC-1.7-0.50						0.50

Figure 2.7 Details of specimens tested by Tran (2010)

2.2 Previous Tests on RC Columns with Plain Rebars

2.2.1 Pandey and Mutsuyoshi (2005)

Pandey and Mutsuyoshi (2005)⁽²⁰⁾ carried out an extensive experimental program employing 15 square columns to examine how the bond of reinforcing bars affects the performance of RC columns. The columns were tested under cyclic loading combined with a low axial force of $0.03f'_cA_g$. The specimens were designed and tested with six different bond conditions, varying from perfect bond to perfect unbond as shown in Figure 2.8. The other test variables included the length of unbonded region, shear to flexural strength ratio and span-to-depth ratio. Details of the test specimens are shown in Figure 2.8. The study found that the failure mode at the ultimate state changed from shear to flexure when reducing the bond strength of the longitudinal rebars. The test results also showed that RC columns with mixed rebars possessed significantly better ductility than RC columns with deformed rebars.

(b) Specimens dimensions

Figure 2.8 Details of specimens tested by Pandey and Mutsuyoshi (2005)**2.2.2 Bousias et al. (2007)**

Bousias et al. (2007)⁽²¹⁾ reported the experimental results of five square columns having plain rebars with the main variable were the length of lap splice. High axial force in the range of $0.41f'_cA_g$ to $0.63f'_cA_g$ was applied. Details of the test specimens are depicted in **Figure 2.9**. Results from the test showed that (i) all specimens exhibited a dramatic drop in resistance after ultimate strength and expressed relatively low deformation capacity. (ii) There was no significant effect of the lap splice length to the behavior of test specimens. (iii) The adverse effect of the axial load level on the post-peak strength degradation was much more evident than any effect of the lap-splicing.

Figure 2.9 Details of specimens tested by Bousias et al. (2007)**2.2.3 Yalcin et al. (2008)**

Yalcin et al.⁽²²⁾ carried out an experimental program consisting of five specimens to study the effect of carbon fiber reinforced polymer (CFRP) columns having plain rebars. Two specimens without CFRP were tested as reference specimens; details are shown in **Figure 2.10**. The columns were tested under a constant axial load of

$0.28f'_cA_g$ and reversed cyclic lateral loading. It was found that columns with continuous rebars produced 10% increment of lateral load capacity.

Figure 2.10 Details of specimens tested by Yalcin et al. (2008)

2.2.4 Verderame et al. (2008, 2010)

Verderame et al. (2008a)⁽²³⁾ carried out an extensive experimental program on six square columns with plain rebars with the focus on the rotational capacity of RC columns designed according to obsolete codes and practices which date back between 1940s and 1970s. Six specimens were tested under monotonic loading and an axial force of $0.12f'_cA_g$ or $0.24f'_cA_g$. Two types of longitudinal reinforcing bars were investigated namely Type A: overlapped rebars at the column base with the lap length of $40d$ and Type B: continuous rebars along the whole column height. The details of the test specimens are shown in **Figure 2.11**. The results showed that (i) the contribution of the rotation of column base was predominant and in a measure of 90% at final stage. (ii) Flexural strength was increased by 50% when the axial force was increased from $0.12f'_cA_g$ to $0.24f'_cA_g$. (iii) Columns with continuous rebars produced 20% higher base rotational capacity than columns with lap splices.

Figure 2.11 Details of specimens tested by Verderame et al. (2008a)

Verderame et al. (2008b)⁽²⁴⁾ tested four columns with plain rebars under cyclic loading. Details of specimens and variables were the same as with those in the columns tested under monotonic loading. The cyclic experiments observed similar responses as in the cases tested under monotonic loading. Cyclic loading resulted in 20% reduction in chord rotation of the corresponding value during monotonic tests.

2.2.5 Arani et al. (2010)

Arani et al.⁽¹⁰⁾ tested four columns having plain rebars with various types of traditional splice detailing. Details of test specimens are shown in Figure 2.12. The specimens were tested under an axial force of $0.15f'_cA_g$ combined with either monotonic loading or cyclic loading. The test results showed that: (i) all specimens responded with limited numbers of flexural cracks, high pinching effects and low residual displacement. (ii) Slip deformation was the major source of total deformation and did not depend on the types of lap-splice. (iii) Splice type did not significantly affect to cyclic stiffness and general behavior of test specimens.

Figure 2.12 Details of specimens tested by Arani et al. (2010)

2.3 Shear Strength Models for RC Columns

Failure of columns is the primary cause of building collapse during earthquakes. Since in the current assessment codes the safety of RC structures are commonly evaluated based on component capacities, analytical approach for accurately estimating shear strength of RC columns is important. The following sections present the available shear strength models for RC columns.

2.3.1 Priestley et al. (1994)

Priestley et al. (1994)⁽²⁵⁾ proposed a shear strength predicting model based on experimental results of eight circular RC columns and six rectangular RC columns. The model expressed the interaction between shear strength and displacement ductility. The shear strength for columns under cyclic load was the summation of strength contributions from concrete, V_c , a truss mechanism (effectively, the transverse reinforcement), V_s , and an arch mechanism associated with axial load, V_p , as follows:

$$V_n = V_c + V_s + V_p \quad (2.1)$$

The concrete component was given by

$$V_c = k\sqrt{f'_c} (0.8A_g) \text{ (Mpa)} \quad (2.2)$$

where the parameter k is a function of the member displacement ductility as shown in **Figure 2.13**.

Figure 2.13 Coefficient k in Priestley et al.'s model (1994)

The contribution of transverse reinforcement to shear strength was expressed based on a truss mechanism using a 30° angle between the diagonal compression struts and the column longitudinal axis. For rectangular-cross-section columns, the truss mechanism component is given by

$$V_s = \frac{A_v f_y d'}{s} \cot 30^\circ \quad (2.3)$$

where d' is the distance measured parallel to the applied shear between centers of the perimeter hoop. A_v , f_y and s are the area, yield strength and spacing of transverse reinforcement, respectively.

The shear strength contribution from axial force mechanism was explained by arch mechanism, shown in **Figure 2.14**, given by

$$V_p = P \tan \alpha = \frac{h-c}{2a} P \quad (2.4)$$

in which h is overall depth; c is the depth of compression zone and a is shear span defined as distance from maximum moment section to point of inflection ($a=L/2$ in Figure 2.14a and $a=L$ for figure 2.14b with L is column clear height). It is noted that the neutral axis depth, c , varies with the curvature at the critical section and, hence, with displacement ductility. The effect of axial tensile load on shear strength was not defined.

Figure 2.14 Contribution of axial load to shear strength (Priestley et al. 1994)

2.3.2 Lynn (2001)

Lynn (2001)⁽⁵⁾ collected a database consisting of a 36 column tests from eight research programs to develop an equation for predicting shear strength. The equation was as follows:

$$V_n = kV_c + \frac{1}{2}V_s = k \left(\frac{0.5A_g\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{0.5\sqrt{f'_c}A_g}} \right) + \frac{1}{2} \frac{A_v f_{yt} d}{s} \quad (\text{Mpa}) \quad (2.5)$$

where V_c and V_s are the concrete shear strength and transverse reinforcement shear strength, respectively; d is the distance from the extreme compression fiber to centroid of tension reinforcement. Other parameters are defined the same as those in the above model. The coefficient k varies from 1 to 0.7 depending on the displacement ductility as shown in **Figure 2.15**. Applying this equation to the experiment database, Lynn found the average ratio between the experimental to the predicted shear strength was 1.04 with a standard deviation of 0.10.

Figure 2.15 Coefficient k in Lynn's model (2001)

2.3.3 Biskinis et al. (2004)

Biskinis et al. (2004)⁽²⁶⁾ analyzed a database consisting of 239 tests on RC specimens of columns and beams to propose an empirical shear strength model for RC columns. This model has been adopted in Eurocode 8 part 3 (2005)⁽²⁷⁾ and expressed as follows (units: MN, m)

When the column is controlled by diagonal tension:

$$V_n = \frac{h-c}{2a} \min(P; 0.55A_g f'_c) + (1 - 0.05 \min(5, \mu - 1)) \left[0.16 \max(0.5; 100\rho_l (1 - 0.16 \min(5; a/h))) \sqrt{\min(f'_c, 40)} bd + V_s \right] \quad (2.6)$$

When the column is controlled by diagonal compression:

$$V_n = \frac{4}{7} (1 - 0.02 \min(5, \mu - 1)) (1 + 1.35n) (1 + 0.45 \cdot 100\rho_l) \sqrt{\min(f'_c, 40)} bz \sin 2\theta \quad (2.6a)$$

where μ is the displacement ductility ratio, θ is the angle between the diagonal and the axis of the column ($\tan \theta = h/2a$), z is the internal arm and taken as $d-d'$, ρ_l is the total longitudinal reinforcement ratio. The model (2.6a) is applicable for squat columns with shear span to depth ratio from 1 to 2, axial load ratio $P/f'_c A_g$ from -0.1 to 0.7 , total longitudinal reinforcement ratio ρ_l from 0.7 to 4% , f'_c from 14.5 to 61 MPa; and displacement ductility ratio μ from 1.4 to 7 .

2.3.4 Sezen and Moehle (2004)

Sezen and Moehle (2004)⁽²⁸⁾ summarized a database consisting of a 51 shear critical columns from cyclic tests to derive a shear strength model. The data base with the range of parameters as: shear span to depth ratio, a/d , from 2 to 4; concrete strength, f'_c , from 17 to 45 MPa; longitudinal reinforcement ratio of 2-4%, the ratio $\rho_v f_y / f'_c$ from 2-4%. where ρ_v is the transverse reinforcement ratio.

The proposed equations for shear strength of RC columns by Sezen and Moehle as follows:

$$V_n = k(V_c + V_s) = k \left(\frac{0.5\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{0.5\sqrt{f'_c} A_g}} \right) 0.8A_g + k \frac{A_v f_y d}{s} \text{ (Mpa)} \quad (2.6)$$

where the coefficient k was taken the same as in Lynn's model. Applying this model to the above database, a mean value of 1.05 (with a standard deviation of 0.15) is obtained for the ratio between the shear strengths determined from experimental and analytical analyses. The result is shown in **Figure 2.16**.

Figure 2.16 Verification of shear strength model by Sezen and Moehle (2004)

2.3.5 Sasani (2004)

Sasani (2004)⁽²⁹⁾ analyzed a database consisting of 89 RC columns to develop a shear strength model for RC columns. In his model, the effect of shear span to depth ratio, compressive load and displacement ductility were taken into account as follow:

$$V_n = 0.22 \left(a_{asp} (1 - 0.88\mu) + 2.9 + \frac{P}{A_g f'_c} \right) \sqrt{f'_c} b d + \frac{2}{3} \frac{A_{sh} f_{yt} d}{s} \text{ (Mpa)} \quad (2.7)$$

$$a_{asp} = \left(3.7 - \frac{M}{Vh} \right) \text{ And if } \frac{P}{f'_c A_g} > 0.42 \rightarrow \frac{P}{f'_c A_g} = 0.42$$

where μ is displacement ductility; other parameters are defined the same as those in the above models. Sasani calculated shear strength for each column and compare with experiment data. The results are shown in **Figure 2.17**.

Figure 2.17 Error in shear prediction by Sasani's model (2004)

2.3.6 ASCE 41 (2007)

ASCE/SEI 41-06 (2007)⁽³⁰⁾ (hereafter abbreviated as ASCE 41) recommends a formula to express the shear strength of RC columns. The equation comes from a regression model on a database of 51 test columns (Sezen and Moehle, 2004), and is given as:

$$V_n = k \frac{A_v f_y d}{s} + \lambda k \left(\frac{0.5 \sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{P}{0.5 \sqrt{f'_c} A_g}} \right) 0.8 A_g \text{ (Mpa)} \quad (2.8)$$

where $k=1.0$ for regions of low ductility demand (displacement ductility < 2), 0.7 for regions of high ductility demand (displacement ductility > 4); k varies linearly for regions of moderate ductility demand. M and V are the moment and shear at the section of maximum moment, respectively, d is the effective depth; and value M/Vd is limited to $2 \leq a/d \leq 4$; λ is 1.0 for normal weight concrete and 0.75 for lightweight concrete.

2.3.7 ACI 318-11 (2011)

In ACI 318-11 (2011)⁽³¹⁾ nominal shear strength of RC columns is derived by a semi-empirical approach as follows:

$$V_n = V_c + V_s \quad (2.9)$$

where V_c is the nominal shear strength provided by concrete and V_s is the nominal shear strength provided by transverse reinforcement.

Shear strength carried by shear reinforcement is given based on 45° truss analogy as

$$V_s = \frac{A_v f_y d}{s} \quad (2.10)$$

Shear strength carried by concrete for the member subjected to axial compression is estimated as

$$V_c = \frac{1}{6} \left(1 + \frac{P}{14A_g} \right) b d \sqrt{f'_c} \quad (\text{Mpa}) \quad (2.11)$$

where b is the width of column section and other parameters are defined the same as those in the above models.

2.3.8 Pan and Li (2013)

Pan and Li (2013)⁽³²⁾ developed a semi-empirical shear strength model based on experimental data of eleven RC columns tested at Nanyang Technological University and existing experimental results of 79 shear-critical RC columns. The model employed the basis of the truss-arch model, which considers both the contributions of concrete and transverse reinforcement to shear strength in the truss model, as well as the contribution of arch action through compatibility of deformation. The proposed shear strength was expressed as:

$$V_n = V_s + k_\mu (V_{ct} + V_a) = k_\mu \left(1 + \frac{K_a}{K_t} \right) V_{ct} + \left(1 + \frac{k_\mu K_a}{K_t} \right) V_s \quad (2.12)$$

where V_s is the contribution of transverse reinforcement to shear strength; V_{ct} is the contribution of concrete to shear strength in truss model; V_a is the shear strength

Chapter II: Literature Review

provided by arch action. k_μ is the factor relating concrete contributions to shear strength in truss model and concrete in arch model to displacement ductility; k_μ is equal to 0.7 for a displacement ductility less than 3.0 and 0.7 for a displacement ductility more than 7.0, with linear interpolation for displacement ductility ranging from 3.0 to 7.0. K_a is the shear stiffness of truss model; K_t is the shear stiffness of arch model;

V_s was calculated based on Modified Compression Field Theory by Vecchio and Collins (1986) as:

$$V_s = \frac{A_{sv} f_{vy} d_v \cot \theta}{s} \quad (2.13)$$

θ is the angle of the inclined strut in cracked concrete with respect to longitudinal axis of column in truss model and expressed in the form of Kim and Mander's model (1999).

$$\theta = \arctan \left(\frac{n \rho_v + \xi \frac{\rho_v b d_v}{\rho_{sl} A_g}}{1 + n \rho_v} \right)^{1/4} \quad (2.14)$$

where n is the ratio between modulus of elasticity of concrete to modulus of elasticity of reinforcing steel; ξ is the factor of boundary condition of the column for calculating θ : $\xi = 0.57$ for fixed-fixed ends and $\xi = 1.57$ for fixed-pinned ends.

V_{ct} was derived based on Bentz et al.'s model (2006) which was a simplified method to calculate the contribution of concrete in truss model. V_{ct} was then presented as:

$$V_{ct} = \frac{\sqrt{B^2 - 4AC} - B}{2A} \quad (2.15)$$

where the solutions for A , B , C are as follows:

$$A = \frac{750}{E_s A_s} \left(1 + \frac{L}{\zeta d_v} \right) \quad (2.16)$$

$$B = \frac{750 V_s}{E_s A_s} \left(1 + \frac{L}{\zeta d_v} \right) - \frac{375 P}{E_s A_s} + 1 \quad (2.17)$$

$$C = -\frac{520 \sqrt{f'_c}}{1000 + s_{ze}} b d_v \quad (2.18)$$

where ζ is boundary condition factor of column, $\zeta = 2$ for fixed-fixed ends and $\zeta = 1$ for fixed-pinned ends ; s_{ze} is the effective crack spacing; for members with stirrups, the stirrups will control the crack spacing, and the term s_{ze} can be simply taken as 300 mm (adopted from CSA 2004).

The shear stiffness of arch model K_t was calculated as:

$$K_t = \frac{n \rho_v E_c b d_v \cot^2 \theta}{1 + n \rho_v \cot^4 \theta} \quad (2.19)$$

The shear stiffness of truss model K_a was calculated as:

$$K_a = E_c b c_a \sin^2 \alpha \cos^2 \alpha \quad (2.20)$$

where α is the inclination of the strut, for column under double curvature $\alpha = (h-x)/L$ and for columns under single curvature $\alpha = (h-x)/2L$.

The neutral axis x was estimated based on an expression from Paulay and Priestley (1975):

$$x = \left(0.25 + 0.85 \frac{P}{f'_c A_g} \right) h \quad (2.21)$$

2.4 Summary

Based on the literature review conducted in this chapter, the remaining shortcomings of the previous studies on the area of existing RC columns can be summarized as follows:

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1. Most tests on existing RC columns had square cross sectional shapes. There is limited test on columns with rectangular section available in previous research. Therefore, the seismic behavior of such columns needs to be further studied.
2. None of available tests on RC columns with plain rebars was conducted under double curvature cyclic bending, so that it may not possible to observe the similar failure patterns to those in real structures including damage around mid-height of the columns. This is because, in single curvature tests, the top end of cantilever (the inflection point in a building column) is restrained and sustains no damage.
3. There is limited comprehensive understanding on the differences in the seismic behavior between columns having plain and deformed reinforcing bars. Thus, the actual seismic performance of RC columns with plain rebars is still far from being well understood.
4. All tests on existing RC columns were conducted in principal directions with regards to column sections. Consequently, the seismic behavior of such columns in non-principal directions remains an open-question.
5. Most analytical work on shear strength modelling based on the concept of diagonal shear failure. It does not express the physical behavior of RC columns those fail by other types of failure especially in the case of vertical splitting failure.

The shortcomings mentioned above provide motivation for the current research.

CHAPTER III: EXPERIMENTAL PROGRAM

Experimental study is indispensable for obtaining insight into fundamental performance of reinforced concrete members and structures. However, in order to simulate the real events on actual structural components it usually requires idealization and simplification for most problems of interest. Within the scope of this research (as presented in Chapter 1) and provided the shortcomings of previous studies (as discussed in Chapter 2), two series of tests including a total of sixteen existing RC column specimens were conducted in this study. This chapter describes the preparations for the experimental program including description of test specimens, test setup, loading and displacement history, instrumentation and measurement. Nominal capacities of test specimens are also predicted herein.

3.1 Design and Construction of Test Specimens

As discussed earlier, two test series were conducted to investigate (i) the behavior of existing columns having plain rebars and (ii) the response of existing columns with deformed rebars under different loading directions. The design approach of the specimens is illustrated in **Figure 3.1**. The design was intended to have specimen geometries, reinforcement content and material properties were similar to the column specimens previously tested by Tran (2010)⁽¹⁹⁾ at Nanyang Technological University, Singapore. The details of these column specimens are representative for old-type RC columns in existing buildings. From obtained test data, two comprehensive studies could be carried out based on two comparisons between each test series and Tran's test results.

Figure 3.1 Specimens design approach

3.1.1 Detail of Specimens Series 1 – Columns with Plain Rebars

Nine half-scale columns in Series 1 were designed for simulated seismic loading tests with details are shown in **Table 3.1** and **Figure 3.2**. More details for each specimen are shown in Appendix A1. The variables of test specimens were axial force, shear span to depth ratio and cross sectional shapes. The test columns were divided into three groups with different cross sectional shapes but similar cross sectional area: square sections (350 x 350 mm), rectangular sections (250 x 490 mm) and wall-like column sections (200 x 610 mm). The clear height of the specimens was either 1200 mm or 1700 mm resulting in a shear span to depth ratio of 1.7, 2.4 for square and rectangular columns or 1.4 for wall-like columns. The column bottom and top bases were designed with heavy reinforcement to simulate stiff foundation or strong floor elements.

Longitudinal reinforcement was plain rebars with a yield stress f_y of 320 MPa. Square and rectangular columns were reinforced with eight R20 rebars (diameter of 20 mm), resulting in a longitudinal reinforcement ratio of 2.05%. Wall-like columns were reinforced with twelve rebars R16 (diameter of 16 mm) with a longitudinal reinforcement ratio of 1.98%. Transverse reinforcement of all test specimens comprised of R6 (diameter of 6 mm) mild steel bars with 135°- bent,

spaced at 125 mm, were characterized by a yield strength f_{yt} of 500 MPa. The transverse reinforcement ratio (defined in Section 2.2.1) was 0.129%, 0.181% and 0.226% for square columns, rectangular columns and wall-like columns, respectively. Details of rebar material properties from rebar tests are shown in **Table 3.1**.

It is noted that the total cross sectional area of transverse reinforcement within spacing s of all test specimens is $A_v = 56.52 \text{ mm}^2$. This value is significantly smaller than the requirement in modern codes. When seismic design is considered, the current design code (ACI 318-11) requires this value cannot be less than the larger of:

$$0.09sh \frac{f_c'}{f_{yt}} \quad \text{and} \quad 0.3sh \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f_c'}{f_{yt}}$$

This formulas result in $A_{vmin} = 198.45 \text{ mm}^2$.

Concrete had normal weight aggregate with maximum specified aggregate size of 10mm and target compressive strength of 25 MPa at 28 days. The actual strength of concrete for each specimen was obtained by the mean of three standard compression tests of concrete cylinders having 150 mm diameter and 300 mm in height. The cylinder tests were conducted at the same day of testing each specimen. The results of cylinder tests are presented in **Table 3.1**.

Table 3.1 Material properties of reinforcing steel

Test Series	Rebar Types	Rebar Diameter (mm)	Yield Strength f_y (f_{yt}) (MPa)	Yield Strain ϵ_y ($\times 10^{-6}$)	Ultimate Strength f_u (MPa)
1	R20	20	348	1683	428
	R6	6	475	2182	526
2	D20	20	545	2523	621
	R6	6	511	2420	648

Table 3.2 Summary of test specimens in Series 1

Specimens	b (mm)	h (mm)	L (mm)	a	A_s (mm ²)	A_v (mm ²)	s (mm)	$\frac{P}{f'_c A_g}$	f'_c (MPa)
SP1-1.7-0.2	350	350	1200	1.7	2512	56.52	125	0.2	29.8
SP2-1.7-0.35	350	350		1.7	2512	56.52	125	0.35	29.2
SP3-2.4-0.2	350	350	1700	2.4	2512	56.52	125	0.2	30.6
SP4-2.4-0.35	350	350		2.4	2512	56.52	125	0.35	28.7
SR1-1.7-0.35	250	490	1700	1.7	2512	56.52	125	0.35	23.3
SR2-1.7-0.5	250	490		1.7	2512	56.52	125	0.5	22.5
SW1-1.4-0.2	200	610	1700	1.4	2412	56.52	125	0.2	21.7
SW2-1.4-0.35	200	610		1.4	2412	56.52	125	0.35	20.2
SW3-1.4-0.5	200	610		1.4	2412	56.52	125	0.5	20.5

Figure 3.2 Detail of specimens in Series 1

3.1.2 Detail of Specimens Series 2 – Columns with Deformed Rebars

Test Series 2 consists of seven half-scale existing column specimens having deformed rebars with the main variable was the directions of cyclic loading. **Table 3.2** and **Figure 3.3** summarize the dimensions and detailing of the test specimens which were similar to those of square and rectangular columns in Series 1. The specimens were divided into three groups: two groups of square columns namely S2.4 and S1.7 for square columns with shear span to depth ratio of 2.4 and 1.7, respectively; one group of rectangular columns namely R1.7 for the columns with an shear span to depth ratio of 1.7.

Longitudinal reinforcement consisted of eight T20 (diameter of 20 mm) deformed rebars, characterized by a yield strength f_y of 545 MPa. The transverse reinforcement of all test specimens insisted of R6 mild steel bars with 135° bent spaced at 125 mm which were characterized by a yield strength f_{yt} of 511 MPa. Details of rebar material properties from rebar tests are shown in **Table 3.1**.

Concrete with target strength of 25 MPa was used. The actual compressive strength was also determined based on average value of three concrete cylinder tests as in Series 1. Results for each specimen are tabulated in **Table 3.3**.

Table 3.3 Summary of test specimens in series 2

Specimen	b (mm)	h (mm)	L (mm)	s (mm)	A_v (MPa)	α (degree)	$\frac{P}{f'_c A_g}$	f'_c (MPa)
S2.4-30	350	350	1700	125	56.52	30	0.2	31.4
S2.4-45	350	350		125	56.52	45	0.2	29.3
S1.7-30	350	350	1200	125	56.52	30	0.2	30.1
S1.7-45	350	350		125	56.52	45	0.2	29.3
R1.7-30	250	490	1700	125	56.52	30	0.35	31.4
R1.7-45	250	490		125	56.52	45	0.35	30.1
R1.7-60	250	490		125	56.52	60	0.35	29.3

Figure 3.3 Detail of specimens in series 2

3.1.3 Specimens Construction

The construction process consisted of several stages, including reinforcing cages, formworks, strain gauging, casting and curing of the specimens. Details of each stage will be provided in the following sections.

Reinforcing Cages

All longitudinal reinforcement and transverse reinforcement were cut and bent by the suppliers. The installation of the strain gauges was only taken 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.

Formwork

Sufficient number of forms was built to cast the specimens simultaneously by two groups with three columns for the first group and four columns for the second one. 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 reinforcement to ensure a clear concrete cover of 25 mm was achieved. **Figure 3.4** shows the formworks with reinforcing cages prior to casting.

Figure 3.4 Formwork with reinforcing cages

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 in a single pour. The specified grade concrete was delivered to the site and then transferred directly to the form by the shoot and mechanically vibrated to remove voids. Concrete cylinders were cast along with the specimens.

After casting, all the specimens were cured for seven days with damp hessian fabrics. The specimens were then transported to the laboratory for test setting-up.

Figure 3.5 Pouring concrete

Figure 3.6 Vibrating concrete

3.2 Experimental Setup

The test setup was designed in such way that it was able to achieve the objectives of this study and to overcome many laboratory constraints due to equipment, testing space and most importantly, safety issues.

A schematic of the loading apparatus is shown in **Figure 3.7** and **Figure 3.8**. The column axial load was applied to the specimens using two vertical actuators (with capacity of 1000 kN) through a L-shaped steel frame. The steel frame was designed to have 250 mm, 400 mm for the width and height, respectively and combined by 30 mm steel plates. This design made the beam capable of transferring forces from actuators to test specimens. The actuators were pinned at both ends to allow rotation in working plane of the columns during the test. Those actuators also contributed to prevent out of plane movement of the specimens. The bottom base of test specimens was fixed to a strong floor by four post tensioned bolts of 60 mm in diameter while the top base was fixed to the L-shaped steel frame by four post tensioned bolts of 30 mm in diameter. Reversible horizontal displacements were applied to the specimen using a 1000 kN capacity actuator 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 strong wall and the L-shaped steel frame.

Figure 3.7 3D schematic view of test setup

Figure 3.8 Photograph of test setup

3.3 Loading and Displacement History

The specimen loading was applied by a combination of a constant vertical force and a cyclic horizontal force. The column axial load was firstly applied to the specimen until the designated level was achieved (as shown in **Table 3.2** and **Table 3.3**). During each test, the axial load was maintained by manually adjusting the vertical actuators after each load step.

The lateral load was applied cyclically through the horizontal actuator in a quasi-static fashion as shown in **Figure 3.7** and **Figure 3.8**. The loading procedure consisting of displacement-controlled steps is illustrated in **Figure 3.9**.

Figure 3.9 Loading procedure

3.4 Instrumentation and Measurement

Instrumentation was installed to measure the global response as well as local deformation of the specimens. A total of around 60 to 70 channels (varied between specimens) were used to collect data measurement from strain gauges, Linear Variable Differential Transducers (LVDTs) and actuators.

3.4.1 Measurement of Strain in Reinforcement

Strain gauges were attached to longitudinal reinforcement as well as transverse reinforcement to measure the local strains in the reinforcing steel. 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. Locations of strain gauges and total number of strain gauges for each specimen are shown in Appendix. All strain gauges were of the KFG type with 5 mm-gauge length, 120 Ω resistance and nominal gauge factor of 2.11. 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. Details on the

preparation and installation of the strain gauges will not be presented in this report as such standard procedures were well documented in various technical reports.

3.4.2 Measurement of Loads

Three independent load cells and an in-built load cell in the computer-controlled actuator were used to measure the applied force on the horizontal actuator as well as the axial pre-loading on the column. The independent load cells were 100 tons in capacity with two full bridge (Poisson) circuits installed. One circuit was directed to a strain indicator and the other to a data logger unit. The independent load cells were connected in series with the hydraulic jack as shown in **Figure 3.7**. Load cells inbuilt were connected to a test system through a signal cable and the load readings were obtained through the Multi-Purpose Test (MPT) program provided by a supplier. The independent load cells were calibrated in a Universal Testing Machine while the in-built load cell of the computer-controlled actuator was factory calibrated.

3.4.3 Measurement of Displacements

The measurement system of displacements consisted of instrumentation to overall horizontal displacement at top of the column, shear and flexural deformation devices located along the height of the specimen and others for checking test setup (movement of bottom base, twisting and out plane movement of the specimen). Data was recorded by an acquisition system including a computer and a data logger connected to switch boxes, to which the measuring devices were linked.

3.4.4 Measurement of Horizontal displacement

The displacements at top of the column were measured during the test procedure by using two LVDTs attached to magnetic stands, which were mounted on steel

frames as shown in **Figure 3.10**. These two LVDTs also could measure twisting as well as out of plane movement of the specimens.

View from right side

View from left side

Figure 3.10 Measurement of horizontal displacement

3.4.5 Measurements of Shear and Flexure Deformations

A total of 23 Linear Potentiometers and LVDTs were attached to each test specimen to track the deformation along the height of the specimens. These devices were attached on 30 mm thick steel plates and were mounted onto 10 mm steel rods embedded in the concrete. 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 Linear Potentiometers arranged in a cross-like manner. It is to be noted that at near failure stages of the test, crushing and spalling of concrete induced false readings on the measuring devices. The affected LVDTs and Linear Potentiometers were then removed as they no longer served any purpose and to prevent them from being damaged.

Calculate 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 3.11**, the rotation of segment 2 due to flexure (θ_{f2}) is given by:

$$\theta_{f2} = \frac{\delta_{2R} - \delta_{2L}}{h_i} \quad (3.1)$$

where δ_{2L} and δ_{2R} are the displacement measured by the left and right transducer at segment 2, respectively; and h_i is the distance between the transducers at segment 2 as illustrated in **Figure 3.11**.

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

$$\phi_2 = \frac{\theta_{f2}}{S_2} \quad (3.2)$$

Where S_2 is the length of segment 2 as illustrated in **Figure 3.11**.

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

$$\delta_{f2} = \theta_{f2} \times x_{f2} \quad (3.3)$$

where x_{f2} is the distance from the center of the column to the center of segment 2 as illustrated in **Figure 3.11**.

Figure 3.11 Evaluation of flexure deformations

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

$$\delta_f = \sum \frac{\delta_{iR} - \delta_{iL}}{h_i} \times x_{fi} \quad (3.4)$$

where subscript 'i' represents the particular segment under consideration

3.1.1 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 3.12**, the average shear distortion at segment 1 (γ_{s1}) is evaluated as follows:

$$\gamma_{s1} = \frac{\delta_{s1} - \delta_{s1}'}{2L_{s1}} \left(\tan \alpha_{s1} + \frac{1}{\tan \alpha_{s1}} \right) \quad (3.5)$$

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 3.12**.

Figure 3.12 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 3.12**.

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

3.5 Nominal Capacities

Nominal capacity of the specimens consists of theoretical yield force, nominal shear strength and theoretical flexural strength.

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 **nominal shear strengths** (V_n) were calculated based on ASCE 41's recommendation:

The **theoretical flexural strengths** (V_u) were estimated using the tested material properties and in accordance with the recommendation provided by ASCE 41.

Nominal Capacities in different loading directions

Shear strength of the specimens in non-principal directions can be predicted based on strengths in two principal directions and findings from Umehara and Jirsa.⁽³³⁾

For square columns, Shear strength is the same regardless of the direction of shear load; for rectangular columns, shear strength in diagonal direction is estimated based on the 2nd-order curve with shear strength from principle direction is known.

Results from these calculations were then tabulated in **Table 3.4**.

3.6 Summary

This chapter describes the experimental preparation program carried out on existing RC columns, including:

1. The construction details of nine columns in test series 1 and seven specimens in test series 2 were described together with details on the loading frame.
2. Special emphasis on the precision in the set-up the test specimens on the loading frame must be taken to ensure the required testing and boundary conditions were achieved.

3. Areas affecting the performance of the specimens were identified. Instrumentation was installed at various locations for measurement and thereafter evaluating the contribution of each factor on the performance of the specimens.
4. Nominal capacities of all test specimens were estimated.

Table 3.4 Nominal capacities of test specimens

Specimens	V_y	V_u	V_n	Specimens	V_y	V_u	V_n
	(kN)	(kN)	(kN)		(kN)	(kN)	(kN)
SP1-1.7-0.2	286.7	328.3	303.1	S2.4-30	182.4	247.1	244
SP2-1.7-0.35	353.3	363.3	354.0	S2.4-45	200.2	236.5	236.5
SP3-2.4-0.2	202.4	231.8	235.9	S1.7-30	267	348.3	308.6
SP4-2.4-0.35	249.4	256.5	266.4	S1.7-45	265	348.3	304.7
SR1-1.7-0.35	251.6	302.4	357.5	R1.7-30	222.2	347.8	297.3
SR2-1.7-0.5	289.5	312.6	389.6	R1.7-45	272.7	270.6	240.3
SW1-1.4-0.2	272.6	327.6	382.3	R1.7-60	179.8	194.9	206.7
SW2-1.4-0.35	315.3	351.8	423.1				
SW3-1.4-0.5	332.5	365.8	471.1				

CHAPTER IV: EXPERIMENTAL STUDY ON EXISTING RC COLUMNS WITH PLAIN REINFORCING BARS

4.1 Introduction

Nine column specimens in Series 1 with details presented in Chapter 3 were tested under a combination of a constant axial load and double curvature bending simulating seismic load. Results of these tests are presented and discussed in this chapter. The performance of the test specimens was examined in terms of cracking patterns, hysteretic response, initial stiffness, shear strength, drift capacities and energy dissipation capacity. The results were then compared with (i) previous experimental data of five columns with similar details but reinforced with deformed rebars⁽¹⁹⁾ and (ii) results predicted using existing assessment code – ASCE 41.⁽³⁰⁾

4.2 Experimental Results and Discussions

Hysteretic responses of the specimens are depicted in **Figure 4.1**. The response of test specimens was highlighted by four performance levels: flexural yielding (*PL1*), maximum shear force (*PL2*), shear failure (*PL3* - shear strength dropped by 20%) and axial failure (*PL4* - when columns were unable to sustain the applied axial force). Crack patterns of test specimens at *PL2* and *PL4* are shown in **Figure 4.2** and **Figure 4.3**, respectively. Generally the hysteretic responses show the pinching effect and reduction of stiffness as well as reduction of shear capacity during repeated cyclic loadings. Except for SR1 all specimens did not reach theoretical yield force and eventually did not reach nominal shear strength and flexural strength. Strain gauges readings indicated that only in SR1 the yielding of longitudinal reinforcement occurred prior to the maximum shear force was recorded.

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Figure 4.1 Hysteretic response of test specimens

(The inclination of V_u and V_n is the result of the P-delta effect.

More details see Appendix A2 on page 171)

Figure 4.2 Crack patterns at maximum shear force

Figure 4.3 Axial failure modes of columns with plain rebars

4.2.1 Square Columns

In specimen SP1, with a shear span to depth ratio of 1.7, vertical cracks were observed along the center rebars around the mid-area of the column height at a drift ratio of 0.33%. Surprisingly, at this stage, there were only two flexural cracks with the length of one-fourth column section height at the bottom end. The appearance of the vertical cracks was the result of the forming and joining of a series of micro inclined cracks crossing each other along the length of the mid-rebars. After reaching the maximum shear force at a drift ratio of 1.67%, the specimen tended to separate into two parts along the vertical cracks. There was a sudden loss of shear capacity right after the maximum horizontal load was attained as seen in the hysteretic response in **Figure 4.1**. This was due to the appearance of a diagonal crack forming with an angle of around 45° to the column axis at the top end of the specimen as depicted in **Figure 4.2**. Unlike diagonal failure observed in other column tests^{(5),(17)} in which the diagonal cracks developed throughout column height to form a diagonal sliding plane, the diagonal crack observed in SP1 only developed from column-base interface to the middle region of column section. After shear failure occurring at a drift ratio of 2.22% the column was still able to sustain the axial force till a large drift ratio of

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4.0% because the diagonal crack did not develop further. The test ended with a vertical cleavage formed throughout the column section, this is defined as *Failure mode 1*. Five transverse reinforcements at the center height were fractured at the end of the test as depicted in **Figure 4.4** which was due to the sliding movement of the column along the cleavage.

Figure 4.4 Typical axial failure modes

Figure 4.5 shows the variation of strains in longitudinal reinforcing bars and transverse reinforcement at different drift ratios for specimen SP1. It is noted that the strain patterns of other specimens was found to be similar to strain patterns of specimen SP1. Under an initial applied axial force of $0.2f'_cA_g$, the average compression strain in longitudinal rebars was about 0.0004 that was approximately 25% of the yield strain. When loaded to a drift ratio of 1.2%, strains in longitudinal rebars increased linearly while strains at all transverse reinforcement remained relatively low at initial strain value. After reaching a drift ratio of 1.2%, strains in longitudinal rebars at different locations varied differently. This variation was due to the development of flexural cracks which led to the re-distribution of stress along the length of main rebars. At the point when the diagonal cracks first appeared, strains in transverse reinforcement at

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both column ends increased significantly. However, there was no yielding of both longitudinal and transverse reinforcement observed till a drift ratio of 3%, beyond the point which shear failure had occurred. Comparing with the case of columns with deformed rebars in Tran's tests (2010)⁽¹⁹⁾ and Tran and Li's tests,⁽³⁴⁾ strain at different locations of longitudinal rebars in the column with plain rebars did not vary significantly as in columns with deformed rebars.

Figure 4.5 Typical strain gauges reading, specimen SP1

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In another short column, SP2, with a higher applied axial force ($P/f'_c A_g=0.35$), vertical cracks were also observed at a relatively early stage, at a drift ratio of 0.4%, along the center rebars. These vertical cracks fully propagated along the column height in next cycle (at a drift ratio of 0.5%). However, dissimilar to SP1, the vertical cracks in SP2 only developed until a drift ratio of 1%. At this stage, diagonal cracks, in the direction of 70° to columns vertical axis, appeared and quickly spread from the top end to the bottom end of the specimen. As expected, under a higher level of axial force the column responded more brittle. Thus the specimen reached its maximum shear force at this point and suddenly lost its shear capacity due to sliding along the diagonal failure plane, this is defined as *Failure mode 2*. It was observed that strain distribution in the reinforcement of SP2 was almost similar to that of SP1 except at a drift ratio of 1.25%. At this stage strain in transverse reinforcement grew rapidly and exceeded the yield strain value. This also resulted from the sliding movement of the diagonal plane.

The longer specimens SP3 and SP4 with a shear span to depth ratio of 2.4 behaved in another manner where flexural response was dominant till the end of the tests. The calculated shear deformations for these two specimens were relatively small at about 8% of the total displacement. Both specimens only developed flexural cracks at column ends before the maximum was reached. SP3 attained maximum shear force at a drift ratio of 1.96% while SP4 reached shear strength at a drift ratio of 1.25% due to 75% higher applied axial force level. Without diagonal cracks, the response of SP3 and SP4 remained stable with gradual reduction of shear capacity even after shear failure. Both specimens failed by crushing of concrete at both column ends and buckling of longitudinal reinforcement in these regions.

4.2.2 Rectangular Columns

The rectangular specimens with a shear span to depth ratio of 1.7, SR1 and SR2, were tested under the applied axial force level $P/f'_c A_g$ of 0.35 and 0.5, respectively. SR1 attained maximum shear force at a drift ratio of 1.2% when some diagonal cracks and crushing of concrete at both ends were observed. Right after that point vertical cracks were formed along the mid-longitudinal rebars, starting from center area and developed to the ends of the specimen. There was also a diagonal crack at mid-height of the column connecting the vertical cracks along two mid-longitudinal reinforcing bars. The test on SR1 was ended by the splitting of vertical cracks, crushing of concrete and buckling of longitudinal reinforcement at both ends of the specimen. This failure mode was similar to Failure mode 1 observed in the test of SP1. Strain gauges reading indicated that no yielding of transverse reinforcement was observed until shear failure attained. The shear deformation was relatively small at about 10% of the total horizontal deformation.

Specimen SR2 reached its maximum shear force with the appearance of two vertical cracks, along the longitudinal rebars at the middle height of column section, starting at a distance of one half column section height to the bottom base. Further loading to shear failure, the vertical cracks continued developing with the patterns similar to those typically observed in Failure mode 1. However, when shear strength dropped about 50%, significant crushing and spalling of concrete were observed at the column bottom end due to the combination of high axial force and high bending moment. This combination reduced the effective area of column cross-section and resulted in buckling of longitudinal reinforcement which led to axial failure. The calculated shear deformation was

relatively small, approximately 10% of total deformation; it was almost the same as that in SR1.

4.2.3 Wall-like Columns

Three wall-like columns SW1, SW2 and SW3 were tested under three different axial force levels: $0.2f'_cA_g$, $0.35f'_cA_g$ and $0.5f'_cA_g$, respectively but failed by the same failure mode as *Failure mode 1*. Observations in all three specimens found that there was no obvious flexural crack at column-base interface when loading to maximum shear force (see **Figure 4.2**). Therefore, the contribution of slip deformation to the total deformation was not significant. It seemed that increasing while the axial force from SW1 to SW3 the diagonal cracks were reduced.

In specimen SW1, with an applied axial force of $0.2f'_cA_g$, diagonal cracks with an angle of about 65° first appeared at a drift ratio of 0.67% and continued developing till a drift ratio of 1% when the specimen reached maximum shear force. Right after this point splitting cracks were observed in the mid-longitudinal rebars. In subsequence cycles the shear capacity gradually reduced which was caused by the opening of the vertical cracks and crushing of concrete at both column ends. At the end of the test concrete spalling formed a damage area along the mid-rebars as shown in **Figure 4.3**.

In specimen SW2 with an applied axial force of $0.35f'_cA_g$, only few diagonal cracks occurred at column ends with an angle of around 40° when loaded to a drift ratio of 0.8%. These cracks did not propagate though the outer column section as in SW1 but only started from the mid-longitudinal rebars. The vertical cracks were formed at a drift ratio of 1% by the crossing of diagonal cracks developed at two different directions corresponding to pulling and pushing of horizontal actuator. This appearance was earlier than that in SW1. The test on SW2 was ended at a drift ratio of 2.27% by the development of the vertical cracks, the

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buckling of longitudinal reinforcement and spalling of concrete at column corners. The crack pattern at final stage of SW2 was similar to the axial failure mode of SW1.

In specimen SW3, tested under a high axial force of $0.5f'_cA_g$, vertical cracks was observed at a quite early stage when loading to a drift ratio of 0.5%. At this stage there were only very minor cracks at the column bottom end while no obvious crack was observed at the column top end. The vertical cracks appeared and propagated at all three middle longitudinal reinforcing bars in subsequence cycles. The column was able to have stable response only till a drip ratio of 1.71% then reached axial failure by buckling of the main rebars at the bottom end.

4.3 Comparisons and Discussions of Test Results

4.3.1 Failure Modes

Most of the columns tests with plain rebars in this study appeared to be controlled by vertical cracks led to splitting failure. It is noted that the vertical cracks have also been observed from previous studies; typically the vertical cracks found in the grain silos which were presented in an article by Collins et al.⁽³⁵⁾ In the case of silos, the concrete walls are expected to bear high circular tensile stress resulting from horizontal pressure exerted by the grain storage while the axial force is not significant. Thus, when the horizontal reinforcement of the silos yielded, vertical cracks occurred in concrete would be inevitable. However, in this study, strain gauge reading indicated that no yielding of transverse reinforcement was observed prior to maximum shear strength was attained. Moreover vertical cracks occurred in the test specimens before its maximum shear force was attained. Therefore horizontal tensile stress would not be the source of vertical cracks in test specimens in this program. The failure mechanism of this type of failure will be explained in detail in the next chapter.

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The columns which failed in *Failure mode I* developed very limited horizontal cracks at column-base interfaces, therefore the contribution of slip deformation to the total deformation was not significant. This is remarkably different from the cases of columns tested under single curvature as reviewed in the introduction part^{(36),(37),(10)} in which the main source of deformation in RC columns with plain rebars was found due to the slip deformation of longitudinal reinforcing bars at connection interfaces. It is noted that, when tested as cantilever, the loading and boundary condition at the top of the columns do not allow forming splitting failure mechanism. Moreover, when plain rebars are used, there is no concentration of stress along the length of the plain rebars as compared to the case of deformed rebars. Therefore, the appearance of flexural cracks will be limited and slip at the column-base-interface is inevitable when the columns with plain rebars are tested under single curvature bending. However, in the tests of RC columns under double curvature, as in this study, vertical sliding along the splitting cracks contributed significantly to hysteretic energy and reduced slip deformation at column-base interfaces.

4.3.2 Shear Strength

Comparisons of backbone curves representing the response of the three groups are shown in **Figure 4.6**. The effects of various test parameters to the performance of specimens are presented in **Figure 4.8-10**. Shear strength of test specimens was normalized as $V_{\max}/(\sqrt{f'_c}A_g)$ and illustrated in **Figure 4.7**. As it has long been recognized that the shear strength benefits from axial force, **Figure 4.7** shows how the shear strength increased with increasing applied axial force. When the axial force ratio (P/f'_cA_g) increased from 0.2 to 0.35, the normalized shear strength of columns having a shear span to depth ratio of 1.7 and wall-like columns increased by approximately 12% while only a slight increase of around

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5% was recorded in square columns with a shear span to depth ratio of 2.4. It is understandable since the effect of axial force on shear strength is reflected on an arch mechanism in which its contribution is reduced with an increment of shear span (or shear span to depth ratio). It also explains, as shown in **Figure 4.7**, why shear strength increased with decreasing shear span to depth ratio. At the axial force ratio of 0.35, the highest normalized shear strength of 0.48 was found in SW2 which was 9% higher than that of SR1. This value fell to 0.40 in specimen SP2 and dropped to 0.30 in SP4.

Figure 4.6 Comparisons of backbone curves of columns with plain rebars

The contribution of axial force to shear strength was not so significant when the test specimens were subjected to high levels of axial compression. It was shown that when the applied axial ratio force was increased from 0.35 to 0.5 the normalized shear force increased by only around 7% in all specimens. This

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observation has been found in few studies,^(38, 39) however it has not been considered in modern design and assessment codes.^{(31),(30)} Therefore, in some cases,^{(38),(40)} the codes may provide un-conservative predictions for shear strength of RC columns. To sum up, axial force has minor effect on the shear strength of columns with high shear span to depth ratio. It also shows evidence that shear strength benefits from axial force if the axial force is not greater than a certain ratio ($P/f_c A_g = 0.35$). This observation well agrees with experimental and numerical results of RC columns with deformed rebars in a previous study.⁽³⁸⁾

Figure 4.7 Effects of test parameters on shear strength

4.3.3 Initial Stiffness

Initial stiffness is normally determined 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. However, this definition would not be applicable to columns in this investigation in which the shear strength does not substantially exceed its theoretical yield force. In this case, therefore, the initial stiffness is 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. **Figure 4.8** shows the effects of axial load, shear span to depth ratio and sectional shape on initial stiffness. It is generally shown that the initial stiffness increased when the applied axial force is increased. The largest change was recorded in short square columns in which the initial stiffness increased by 23.7% from 25.8 kN/mm in SP1 to 31.9 kN/mm in SP2. The change was smaller to 20.7% from SP3 to SP4 and at 15.5% from SR1 to SR2. The most remarkable result was found in wall-like columns in which initial stiffness increased very slightly, 3.6% and 6.3%, when the axial force ratio was increased from 0.2 to 0.35 and 0.5.

Figure 4.8 Effects of test parameters on initial stiffness

4.3.4 Drift Capacities

The effects of test parameters on the drift capacities (drift ratio at maximum shear force, at shear failure and at axial failure) are shown in **Figure 4.9**. Overall, drift capacities were decreased when the applied axial force was increased. The drift ratio at maximum shear force dropped sharply in the group of square columns when the axial force ratio increased from 0.2 to 0.35. The drift degradation was 48.2% from SP1 to SP2 and 56.8% from SP3 to SP4. Due to the difference in failure mode with the sudden loss of shear capacity in SP2, the gap in drift

capacities between SP1 and SP2 remained at shear failure and axial failure. Apart from a slight slope between SP3 and SP4 at shear failure, the trends of drift capacities for other specimens remained consistent at different stages of the tests where wall-like columns showed the lowest drift capacities.

Figure 4.9 Effects of test parameters on drift capacities

4.3.5 Energy Dissipation Capacity

Energy dissipation capacity was defined as the area under the load-displacement hysteresis curves. Effects of test parameters on Energy dissipation capacity at different performance levels are shown in **Figure 4.10**. The observed trends were almost similar to that of drift capacities. The total energy dissipated (at axial failure) depends on not only shear span to depth ratios and applied axial force but also sectional shapes. With the same cross sectional shape, decreasing the shear

span to depth ratio from 2.4 to 1.7 the total energy dissipated decreased by 68% when axial force ratio was 0.2 (from SP3 to SP1) and by 275% when axial force ratio was 0.35 (from SP4 to SP2). Having the same shear span to depth ratio and axial load ratio, the rectangular specimen, SR1, produced much higher energy dissipation capacity compared to that of square specimen – SP2.

Figure 4.10 Effects of test parameters on energy dissipation capacity

4.4 Comparisons with Columns having Deformed Rebars

Five specimens in this study were designed to be similar to columns with deformed rebars in Tran’s tests⁽¹⁹⁾ in terms of specimens’ geometries, reinforcement ratios and concrete strength. All tests in both studies were conducted using the same test set-up. The columns with deformed rebars, including SC-1.7-0.2, SC-1.7-0.35, SC-2.4-0.2, RC-1.7-0.35 and RC-1.7-0.5, were compared with columns with plain rebars SP1, SP2, SP3, SR1 and SR2,

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respectively (Refer to Section. 2.1.6 & Table 2.1 for the characteristics of the Tran's specimens⁽¹⁹⁾). Comprehensive comparisons are then possible to find out how the surface characteristic of longitudinal reinforcement influences the seismic response of RC columns.

Figure 4.11 Comparisons of backbone curves between columns with plain and deformed rebars

4.4.1 Comparisons of Overall Response and Failure Modes

Comparisons of backbone curves between columns with plain rebars and deformed rebars are shown in **Figure 4.11**. Crack patterns of columns with deformed rebars are presented in **Figure 4.12** and **Figure 4.13**. Most columns with plain rebars performed in flexural deformations dominant mode. Even those specimens designed with light transverse reinforcement exhibited sufficient flexural deformations, while shear deformation was relatively small at 10-15% of total deformation when maximum shear force was attained. On the other hand, significant shear deformations were recorded in columns with deformed rebars, the ratio in between 20-40%. It was because all columns with deformed rebars experienced substantial diagonal cracks during its elastic range while columns with plain rebars developed vertical cracks from an early stage. When the vertical cracks occurred, the external energy was mainly dissipated by the splitting of concrete through sliding between two separated parts along the cracks. Thus the longitudinal and transverse reinforcement did not yield even when shear failure occurred in columns with plain rebars. On the contrary the diagonal cracks developed in columns with deformed rebars as a result of combined shear and flexural stresses were inclined at an angle to the axis of the columns. These cracks resulted in yielding of transverse reinforcement before the columns reached the maximum shear force. The columns with deformed rebars also developed flexural yielding prior to the maximum shear force attained. Because of low bond condition between reinforcement and surrounding concrete in columns with plain rebars, the tensile force carried by longitudinal reinforcement, if any, was distributed more regularly along its length than in the case of columns with deformed rebars. Therefore, in columns with plain rebars, the strain along longitudinal rebars did not vary significantly and cracks spacing was larger than in columns with deformed rebars.

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From **Figure 4.12** and **Figure 4.13** it can be seen that only the rectangular column under applied axial force of $0.5f_cA_g$ failed with the same failure mode. Specimens with plain rebars SP1, SP3 and SR1 failed by the forming of vertical cracks while specimens with deformed rebars SC-1.7-0.2, SC-2.4-0.2 and RC-1.7-0.2 failed by the development of diagonal shear plan. The reverse change was found in specimen SP2 and SC-1.7-0.35. Therefore, it is evidence that the surface of longitudinal reinforcement might changes the failure mechanism of RC columns.

Figure 4.12 Crack patterns at maximum shear force of columns with deformed rebars

Figure 4.13 Axial failure of columns with deformed rebars

4.4.2 Comparisons of Initial Stiffness

As mentioned in the preceding section, the initial stiffness of columns with deformed rebars might be calculated based on the yielding point of longitudinal reinforcement. However, most columns with plain rebars did not develop flexural yielding therefore, to be consistent, initial stiffness of all columns was calculated based on the first point of 80% shear strength obtained. The comparison of initial stiffness is tabulated in **Table 4.1**. Interestingly, most columns with plain rebars possess higher initial stiffness compared to columns with deformed rebars. The biggest difference was found in rectangular columns, where plain rebars resulted in a stiffness improvement of 70.6% and 108% on SR1 and SR2, respectively. The comparisons also showed that for short columns (all specimens with shear span to depth ratios of 1.7) deformed reinforcement did not produce higher initial stiffness. The reason was because the flexural cracks occurred in columns with deformed rebars reducing its initial stiffness gradually. It was observed that all specimens with a shear span to depth ratio of 1.7 did not develop much flexural cracks; instead those specimens tend to fail by splitting cracks.

4.4.3 Comparisons of Shear Strength

The comparisons of normalized shear strengths are summarized in **Table 4.1**. The results show that in all cases, the columns with plain rebars produced shear strength significantly less than the columns with deformed rebars. The reduction of shear strength was almost similar in the three square columns and at approximately of 33% while it was around 21% in rectangular columns. Seemingly these reductions were not sensitive to the change of shear span to depth ratio or applied axial force but relatively sensitive to the cross sectional shape of the columns.

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Under the combination of axial force and shear force, the columns may form potential splitting cracks along the length of the mid-rebars. As observed in this study, the shear strength of the test specimens was likely to be controlled by its splitting strength. And naturally plain reinforcing bars yield lower bond capacity than deformed rebars, leading to lower splitting strength. Therefore, the columns with plain rebars predictably produce lower shear strength than columns with deformed rebars.

Table 4.1 Comparisons between columns with plain rebars and columns with deformed rebars

Specimen	$\frac{P}{f'_c A_g}$	$\frac{V_{max}}{A_g (f'_c)^{0.5}}$	I_m	δ_m	δ_s	δ_a	Failure mode	Energy dissipation (kNmm) at		
			(kN/mm)	(%)	(%)	(%)		PL2	PL3	PL4
SP1-1.7-0.2	0.2	0.36	25.8	1.67	2.22	4.00	1	19.0	22.0	51.4
SC-1.7-0.2	0.2	0.47	22.4	1.30	1.43	1.82	2	5.0	7.5	13.6
SP2-1.7-0.35	0.35	0.4	31.9	1.13	1.43	1.52	2	10.2	14.1	18.6
SC-1.7-0.35	0.35	0.54	25.6	1.26	1.44	1.56	1	5.5	7.5	9.5
SP3-2.4-0.2	0.2	0.29	12.8	1.96	2.87	4.06	1	25.8	57.6	86.3
SC-2.4-0.2	0.2	0.38	14.2	1.44	1.98	2.82	2	10.5	22.0	34.0
SR1-1.7-0.35	0.35	0.45	31.6	1.02	1.81	2.80	1	13.2	33.6	59.0
RC-1.7-0.35	0.35	0.54	15.2	1.45	1.65	2.02	2	11.0	21.5	25.0
SR2-1.7-0.5	0.50	0.47	36.5	0.85	1.36	1.71	1	14.0	23.0	34.0
RC-1.7-0.5	0.50	0.57	21.4	1.44	1.67	1.80	1	11.5	15.0	22.5

(Note: δ_m , δ_s , δ_a = drift ratio at maximum shear force, shear failure and axial failure)

The previous test conducted by Pandey and Mutsuyoshi⁽²⁰⁾ also found that the specimen with plain rebars (specimen C6) produced 22% lower shear strength than the specimen with deformed rebars (specimen C1). Faella et al⁽⁴¹⁾ conducted a test series on RC columns with plain rebars and deformed rebars and with retrofitted columns and un-retrofitted columns. The results showed that in all cases the columns with deformed rebars possessed higher shear strength than the columns

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with plain rebars. The differences ranged from 20% to 28% in un-retrofitted columns and 33% to 43% in retrofitted columns. All these studies confirm that columns with deformed rebars possess higher shear capacity than columns with plain rebars.

4.4.4 Comparisons of Drift Capacities

Generally, columns with deformed rebars developed flexural yielding before reaching maximum shear force while columns with plain rebars reached shear strength before yielding of longitudinal reinforcement. Under lower applied axial force, plain longitudinal reinforcing rebars resulted in more ductile response in RC columns. In particular, specimen with plain rebars SP1 reached maximum shear force at a drift ratio of 1.67%, while the specimen with deformed rebars SC-1.7-0.2 obtained maximum shear force at a drift ratio of 1.3%, which was 30% earlier. Similarly, with a shear span to depth ratio of 2.4, the column with plain rebars (SP3) attained maximum shear force at the drift ratio around 50% larger than that of column with deformed rebars (SC-2.4-0.2). When the applied axial force was increased to $0.35f'_cA_g$, the recorded drift ratios at the maximum shear force were almost identical in both columns with plain rebars and columns with deformed rebars. The results were around 1.56% for specimens SR1, RC-1.7-0.35 and 0.8% for the specimens SP2, SC-1.7-0.35. For high axial force, $0.5f'_cA_g$, the trend was reversed. The columns with deformed bar reached the maximum shear force later than columns with plain rebars by about two times the drift ratio.

The drift ratio at axial failure of columns with plain rebars was significantly higher than those of columns with deformed rebars, when the applied axial force was lowest at $0.2f'_cA_g$. The results are tabulated in **Table 4.1**, showing that for the short columns, with a shear span to depth ratio of 1.7, the column with plain bar SP1 reached axial failure at 4.0% drift while the column with deformed rebars,

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SC-1.7-0.2, reached at a drift ratio of 1.82%. The longer columns, with a shear span to depth ratio of 2.4, were failed at a drift ratio of 4.1% and 2.82% for the column with plain rebars and deformed rebars, respectively. When the applied axial force level was higher than $0.35f_c A_g$, the drift ratio at axial failure tended to be similar in both columns with deformed rebars and plain bars.

4.4.5 Comparisons of Energy Dissipation Capacity

Energy dissipated corresponding to each performance level of all specimens is tabulated in **Table 4.1**. Generally, energy dissipated by columns with plain rebars was significantly higher than that of columns with deformed rebars at all performance levels. At the point when the maximum shear force was attained, the energy dissipated by square columns with plain rebars was as high as double that of square columns with deformed rebars, but only slight difference was found at rectangular columns. The discrepancy in energy dissipated by columns with deformed rebars and columns with plain rebars was even increased when the columns were loaded to shear failure. When axial failure occurred, although it failed with a same failure mode, SR2 still dissipated an amount of energy 50% higher than RC-1.7-0.5. In the other four pairs, with the observed failure mode changing, the difference was more remarkable. Energy dissipation was enhanced by 278%, 154%, 136% and 95% in columns with plain rebars SP1, SP3, SR1 and SP2, respectively.

4.5 Comparisons with ASCE 41

In this part, the backbone curves obtained from the experimental results of all test specimens are compared with assessment results from ASCE 41 guidelines' models.⁽³⁰⁾ According to ASCE 41, the force-displacement relationship follows the general trend as shown in **Figure 4.14** for columns whose strengths are limited by flexural strength where shear force is normalized with respect to its

yield force. The deformation indexes (a , b) as illustrated in **Figure 4.14** are defined as flexural plastic hinge rotations that depend on column axial load, nominal shear stress and details of columns. The index c is equal to 0.2.

Figure 4.14 Generalized force-displacement relationship in ASCE 41

It should be noted that the shear strength equation of ASCE 41 comes from a regression model on a database of 51 lightly reinforced concreted columns⁽²⁸⁾ and does not employ the effect of plain reinforcing bars.

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(Figure 4.15 – continued)

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Figure 4.15 Comparison of backbone curves between experiment and ASCE 41

Figure 4.15 and **Table 4.2** compare the backbone curves of the test specimens with analytical predictions by ASCE 41 provisions. It is shown that ASCE 41 significantly over-estimates the initial stiffness of columns with plain rebars. The stiffness from the guidelines is not consistent, varying from 3 times to 13 times higher than those from the test. In turn, however, the drift at maximum shear force and at axial failure is under-estimated by ASCE 41. On the other hand the shear strength recorded by the tests was consistently smaller than estimated results from ASCE 41 by around 30%. The comparison between columns with deformed rebars and plains rebars tests also showed that shear strength could be improved by 20-30% when replacing the plain rebars by deformed longitudinal reinforcement. Moreover, previous experimental studies⁽³⁸⁾ have shown that ASCE 41 provides good estimations of shear strength for RC columns with deformed rebars. Therefore, it is recommended to multiply with a modification factor k_m , $k_m=1.0$ for columns with deformed longitudinal reinforcement, $k_m=0.70$ for columns with plain longitudinal reinforcement, the shear strength when evaluating for columns with plain rebars.

$$V_n = k k_m \frac{A_v f_{yt} d}{s} + \lambda k k_m \left(\frac{0.5 \sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{P}{0.5 \sqrt{f'_c} A_g}} \right) 0.8 A_g \text{ (MPa)} \quad (4.1)$$

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Verifications from the current tests and previous tests on columns with plain rebars are shown in **Table 4.3**. The results show that the mean difference between the modified shear strength formula and results from 19 tests of columns with plain rebars is almost 0.00% with a 12.9% Coefficient of Variation. This is a significant improvement compared with the shear strength predictions calculated by the original ASCE 41's formula (Eq. (4.1)).

Table 4.2 Comparisons between the current test results and prediction by ASCE 41

Specimens	Initial stiffness (kN/mm)			Drift at V_{max} (%)			Drift at Axial failure (%)			Shear strength (kN)		
	K_{test}	K_{ASCE}	$\frac{K_{test}}{K_{ASCE}}$	δ_{test}	δ_{ASCE}	$\frac{\delta_{test}}{\delta_{ASCE}}$	δ_{test}	δ_{ASCE}	$\frac{\delta_{test}}{\delta_{ASCE}}$	V_{test}	V_{ASCE}	$\frac{V_{test}}{V_{ASCE}}$
			K_{ASCE}			δ_{ASCE}			δ_{ASCE}			δ_{ASCE}
SP1-1.7-0.2	25.8	224.2	0.12	1.67	0.71	2.22	4.00	1.55	2.59	238.3	303.1	0.79
SP2-1.7-0.35	31.9	222	0.14	1.13	0.59	1.90	1.52	1.32	1.15	263.2	354.0	0.74
SP3-2.4-0.2	12.8	79.9	0.16	1.96	0.80	2.46	4.06	1.63	2.49	193.3	235.9	0.82
SP4-2.4-0.35	15.4	77.4	0.20	1.25	0.69	1.80	3.06	1.43	2.14	198.0	266.4	0.74
SR1-1.7-0.35	31.6	136.7	0.23	1.02	0.56	1.81	2.80	1.3	2.16	263.0	357.5	0.74
SR2-1.7-0.5	36.5	110.8	0.33	0.85	0.46	1.87	1.71	1.16	1.48	278.0	389.6	0.71
SW1-1.4-0.2	24.4	203.6	0.12	1.05	0.66	1.59	3.31	1.49	2.22	247.8	382.3	0.65
SW2-1.4-0.35	25.3	196.4	0.13	0.91	0.52	1.74	2.27	1.26	1.81	266.0	423.1	0.63
SW3-1.4-0.5	26.9	169.8	0.16	0.76	0.43	1.77	1.71	1.13	1.51	289.0	471.1	0.61
Ave			0.177			1.907			1.950			0.715
CV			0.387			0.240			0.252			0.099

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Table 4.3 Modified shear strength equation of ASCE 41 for columns with plain rebars

Specimen	<i>L</i> (mm)	<i>b</i> (mm)	<i>h</i> (mm)	<i>f_c</i> (Mpa)	<i>f_{st}</i> (Mpa)	<i>A_v</i> (mm ²)	<i>s</i> (mm)	<i>ρ_v</i> (%)	<i>P</i> (KN)	<i>V_{test}</i>	<i>V_n</i> (kN) [ASCE41]	<i>V_n</i> (kN) [Modified]	<i>V_{test}</i> <i>V_{ASCE}</i>	<i>V_{test}</i> <i>V_{Mod}</i>
Current Test														
SP1-1.7-0.2	1200	350	350	29.8	475	56.5	125	0.129	730.1	238.3	303.1	212.1	0.79	1.12
SP2-1.7-0.35	1200	350	350	29.2	475	56.5	125	0.129	1252.0	263.2	354.0	247.8	0.74	1.06
SP3-2.4-0.2	1700	350	350	30.6	475	56.5	125	0.129	749.7	193.3	235.9	165.1	0.82	1.17
SP4-2.4-0.35	1700	350	350	28.7	475	56.5	125	0.129	1230.5	198.0	266.4	186.5	0.74	1.06
SR1-1.7-0.35	1700	250	490	23.3	509	56.5	125	0.092	999.0	263.0	357.5	250.2	0.74	1.05
SR2-1.7-0.5	1700	250	490	22.5	509	56.5	125	0.092	1378.1	278.0	389.6	272.7	0.71	1.02
SW1-1.4-0.2	1700	200	610	21.7	509	56.5	125	0.074	529.5	247.8	382.3	267.6	0.65	0.93
SW2-1.4-0.35	1700	200	610	20.2	509	56.5	125	0.074	862.5	266.0	423.1	296.2	0.63	0.90
SW3-1.4-0.5	1700	200	610	20.5	509	56.5	125	0.074	1250.5	289.0	471.1	329.8	0.61	0.88
Arani et al ⁽¹⁰⁾														
WOS-M	1500	250	250	23.9	370	100.5	200	0.201	224.1	49.8	88.4	61.9	0.56	0.80
WOS-C	1500	250	250	22.9	370	100.5	200	0.201	214.7	62.1	87.0	60.9	0.71	1.02
SOS-M	1500	250	250	24.0	370	100.5	200	0.201	225.0	65.0	88.5	62.0	0.73	1.05
HOS-C	1500	250	250	24.8	370	100.5	200	0.201	232.5	65.0	89.6	62.7	0.73	1.04
Yalcin et al ⁽²²⁾														
L0C0	3200	200	400	16.0	360	100.5	300	0.084	352.0	77.0	92.3	64.6	0.83	1.19
L50C0	3200	200	400	17.0	360	100.5	300	0.084	374.0	75.0	94.4	66.1	0.79	1.14
Faella et al ⁽⁴¹⁾														
M5-S	3400	300	300	26.4	346	100.5	200	0.167	332.6	51.2	85.9	60.2	0.60	0.85
C3-S	3400	300	300	25.7	346	100.5	200	0.167	323.8	52.7	85.2	59.6	0.62	0.88
C16-S	3400	300	300	27.5	346	100.5	200	0.167	990.0	81.5	106.7	74.7	0.76	1.09
C18-S	3400	300	300	13.5	346	100.5	200	0.167	486.0	42.3	82.1	57.4	0.52	0.74
Ave													0.70	1.00
CoV													0.129	0.129

4.6 Summary

An experimental program was carried out on nine reinforced concrete columns with plain longitudinal reinforcing bars under simulated gravity and seismic forces. The experimental results were analyzed and compared with those of columns with deformed reinforcing bars. The test results were also used to evaluate the current assessment code ASCE 41. The conclusions can be drawn as follows:

1. All columns with plain rebars failed with the development of vertical cracks forming along the longitudinal reinforcing bars located at the middle height of column sections. Vertical cracks developed earlier on the columns with higher applied axial load. This type of cracks did not substantially affect the initial stiffness. Unlike columns with plain rebars tested as cantilevers, the columns tested under double curvature bending did not experience remarkable slip deformation. Instead, the deformation resulting from sliding movements of the column body along splitting cracks was believed to contribute significantly to the energy dissipation capacity.
2. The effects of the shear span to depth ratio, level of axial force, cross sectional shape on the performance of columns with plain rebars were similar to those on columns with deformed rebars. Most remarkably, shear strength of columns with plain rebars benefited from increasing the axial force up to $0.35f'_cA_g$.
3. Unlike columns with deformed rebars, most columns with plain rebars did not develop flexural yielding and also did not experience the yielding of transverse reinforcement before the maximum shear force was reached. Columns with deformed rebars were found to possess higher shear strength than columns with plain rebars, ranging from 22% to 33%, depending on cross sectional shapes but regardless of the levels of axial force or shear span to depth ratio. However, columns with plain rebars were generally shown to have higher initial stiffness

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and significantly higher energy dissipation capacity. Under moderate applied axial compressive force levels, the failure mechanism could change from diagonal shear failure to splitting failure when deformed rebars were replaced by plain rebars.

4. The method to assess the performance of RC columns in existing buildings, in ASCE 41, inconsistently and substantially over-estimates the initial stiffness of RC columns with plain rebars. Consequently, drift capacities are not well predicted. Furthermore, the guideline gives an overestimate of shear strength for columns with plain rebars. Results from the tests suggest multiplying with a modification factor of 0.70 to the shear strengths estimated from Equation (6-4) in ASCE 41 when evaluating columns with plain rebars.

CHAPTER V: MECHANISM OF SPLITTING FAILURE AND PROPOSAL OF A NEW SHEAR STRENGTH MODEL

5.1 Introduction

Results from the tests of specimens in Series 1 as presented in Chapter 4 have shown a major proportion of tested RC columns failed by splitting failure. This type of failure is described as the formation of vertical cracks along the column height. The failure has attracted attention of researchers since 1970s when researchers in Japan found that nearly one-third of 260 specimens failed in splitting along longitudinal reinforcement (Higashi et al. 1974). An illustration of this phenomenon in a real event of an earthquake is shown in **Figure 5.1**, in which RC columns experienced splitting failure without remarkable signs of flexural and shear cracks. Splitting cracks propagating along the main reinforcing bars prevents the development of diagonal compression struts. As a result, the shear strength and the post-elastic response of the columns will be affected, especially for the case of short columns. Therefore this phenomenon needs to be investigated.

Figure 5.1 Splitting failure observed in Grant Thornton Building, Christchurch, New Zealand earthquake 2011 (Courtesy of Nigel Priestley)

Suenaga and Ishimaru 1974⁽⁴²⁾ proposed the “Wedge Theory” for squat plain concrete columns which were based on the yielding condition of concrete. In this method, the mechanism of splitting failure was assumed by forming yield lines ABCD as shown in **Figure 5.2**. However, since this model ignored the stress carried by transverse reinforcement and shear stress acting along the failure plane (BC line in **Figure 5.2**), the formation of vertical cracks could not be explained. Studies by Ichinose⁽⁴³⁾ and Lura et al.⁽⁴⁴⁾ demonstrated that splitting failure is related to bonding failure and occurred when the local bond of longitudinal reinforcement exceeded the bond stress capacity. While this might be valid, the studies did not explain the overall performance of the whole column body. Kaminosono⁽⁴⁵⁾ explained the formation of vertical cracks using an idealized deformed shape of the column as shown in **Figure 5.3**. In that study, the author assumed the columns to be in a fully developed tensile hinge zones prior to the formation of vertical cracks. However, observations from earthquake and experiment have shown that the vertical splitting cracks developed at a very small horizontal displacement and did not require the formation of flexural cracks or a plastic hinge zone. Moreover, in the model shown in **Figure 5.3**, an external force Q had resulted in compression in transverse reinforcement which was not reasonable because the stirrups normally carry tensile stress. This has also been confirmed by strain gauge measurements in other columns tests.^{(34), (38)}

Figure 5.2 Failure mechanism and stress state in Wedge theory (Suenaga and Ishimaru 1974)

Figure 5.3 Idealized deformation and assumed forces in Kaminosono's model

Although the behavior of RC columns in shear has been studied for years, engineers are still unable to accurately determine the shear strength of RC columns. Most shear strength models available in literature and guidelines (ACI 318-11,⁽³¹⁾ Sezen and Moehle (2004),⁽²⁸⁾ ASCE 41 (2007),⁽³⁰⁾ Biskinis et al. (2004),⁽²⁶⁾ EN 1998:3 (2005)⁽²⁷⁾) generally refer to diagonal failure and estimate the contribution of transverse reinforcement according to the Ritter-Morsch truss analogy. The shear strength models based on diagonal failure do not reflect the true behavior of columns in splitting failure. This study aims to clarify the mechanism of splitting failure mode of RC columns through the evaluation of splitting stress along potential vertical planes. Based on experimental observations, a new model for predicting the shear strength of RC columns that experience splitting failure will be developed. Further attempts are then made to extend the applicability of the proposed model.

5.2 Observed Splitting Failure

From experimental results in Chapter 4, it was observed that vertical cracks formed in all specimens at an early stage without an obvious sign of flexural crack and shear cracks which are similar to the case in **Figure 5.1**. The typical response (specimen SP1-1.7-0.2) discussed in previous chapter is illustrated in **Figure 5.5**.

Figure 5.4 Typical hysteretic response and crack development of test specimen

Splitting failure was also observed in the tests of columns with deformed longitudinal reinforcement conducted by Tran and Li⁽³⁴⁾ and Pham and Li⁽⁴⁶⁾ using the same test setup and loading procedure as the test program in this study. The crack patterns of typical columns (rectangular specimens tested under axial load of 0.05 and $0.5f'_cA_g$) are shown in **Figure 5.6**. Comparing those specimens with the specimens which did not fail by splitting failure (specimens in the study of Tran (2010)⁽¹⁹⁾), it was found that columns that did not fail in splitting failure had a higher shear strength, but much smaller drift capacities (drift ratios at shear failure and axial failure).

The observations of failure mechanism were similar to the failure of specimen SP1-1.7-0.2 described above. It was found that specimens failed by splitting failure were short columns with a shear span to depth ratio smaller than 2.5 RC columns with vertical cracks or splitting failure in other available studies are tabulated in **Table 5.1**.

Figure 5.5 Typical failure modes of test specimens

Figure 5.6 Vertical cracks observed at maximum shear force of deformed rebars columns tested by Tran (2010)

5.3 Mechanism of Splitting Failure

From experimental observation, most of the specimens failed with the formation of vertical cracks. At the end of the test, one vertical plane fully developed at the mid-height of the columns separating the column body into two parts. It was observed

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that the vertical cracks appeared at an early stage (low drift ratio) without yielding of transverse reinforcement (indicated by strain gauges). Therefore, the formation of vertical cracks could be due to the shear stress acting along the potential splitting crack. Under double curvature moment and axial force as shown in **Figure 5.7a**, the column formed two compression zones at its ends. The applied axial force and shear force will be transferred from the compression zone at the top end through the potential splitting plane to the compression zone at the bottom end. The yield lines were possibly formed as depicted in **Figure 5.7b**. In this regard, the column might fail in one of the following mechanisms:

Figure 5.7 Splitting failure mechanism

- Failure at compression zone due to concrete crushing;
- Yielding of transverse reinforcement;
- Splitting failure at potential sliding crack.

From **Figure 5.7b**, the applied horizontal force was transferred by normal stress to vertical cracks along the mid-bar; the applied axial force was transferred from the

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top-end to the bottom end of the column through friction stresses along the surface of the splitting cracks; The bending moment at the ends was modelled by a couple of forces: A compression force F ($F=M/d$) acting at the center of the compression zone; A tensile force T_s acting on the tensile longitudinal reinforcement.

The resultant forces are shown in **Figure 5.7c**.

The sliding shear force in the compression zone S_1 is the total shear stress transferred through the diagonal plane:

$$S_1 = \tau_{s1} b \frac{h}{2 \sin \alpha} \quad (5.1)$$

Shear stress (or sliding shear stress) acting on potential sliding plane is calculated as:

$$S_2 = \tau_{s2} b (L - h \tan \alpha) \quad (5.2)$$

where τ_{s1} and τ_{s2} are the average shear stresses on the diagonal and vertical plane, respectively, b is the column section width, h is column section depth, L is column height, α is the angle of the compression strut at column ends.

The compression force transferred through the compression zone can be calculated:

$$C_c = c_{s1} b \frac{h}{2 \sin \alpha} \quad (5.3)$$

Neglecting the contribution of concrete in tension, the tensile stress on the vertical plane is carried out by transverse reinforcement. The resultant force T_s is, therefore, calculated as:

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$$T_s = \tau_v \frac{L-h}{s} A_v \quad (5.4)$$

where s , A_v and τ_v are the spacing, area and tensile stress of transverse reinforcement, respectively.

Equilibrium in the horizontal direction is given as:

$$V = 2S_1 \cos \alpha + T_s - 2C_c \sin \alpha \quad (5.5)$$

where V is the applied shear force, S_1 is the shear force transferred through the compression zone, T_s is the shear force carried by transverse reinforcement (ignoring the tensile force contributed by concrete in tensile region), C_c is the compression force acting at the compression zone.

Equilibrium in the vertical direction can be given as:

$$P = 2S_1 \sin \alpha + S_2 + 2C_c \cos \alpha \quad (5.6)$$

Where P is the applied axial force, S_2 is the splitting shear force transferred through splitting cracks.

From Eq. (5.5) and Eq. (5.6) we have:

$$V = P + T_s - S_2 + 2S_1(\cos \alpha - \sin \alpha) - 2C_c(\cos \alpha + \sin \alpha) \quad (5.7)$$

From the experimental observations (presented in the previous part) the vertical cracks generally developed at a point forming an angle of 45 degree to the corner of the column therefore the angle α can be reasonably assumed as 45°, then Eq. (5.7) can be rewritten as:

$$V = P + T_s - S_2 - 2\sqrt{2}C_c \quad (5.8)$$

The shear force carried by transverse reinforcement is calculated as:

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$$T_s = k_1 \frac{L-h}{s} A_v f_{yt} \quad (5.9)$$

where L is the column height; s , A_v and f_{yt} are the spacing, area and yield stress of transverse reinforcement, respectively; k_1 is a factor expressing the level of tensile stress in transverse reinforcement $k_1 \leq 1$.

The splitting shear force S_2 was taken by the aggregate interlock and dowel action developed in transverse reinforcement across the vertical failure plane. However, Park and Paulay⁽⁴⁷⁾ suggest that dowel action is not a major component of shear-resisting mechanism across cracked interfaces at acceptable shear displacements. It is, therefore, reasonable to assume that when vertical cracks occurred (at low drift ratio as observed) S_2 was mainly resisted by aggregate interlock of concrete and can be written as:

$$S_2 = k_2 f_{ct} A_{sp} \quad (5.10)$$

where k_2 is a factor expressing the level of sliding shear stress in concrete along splitting crack $k_2 \leq 1$, A_{sp} is the area of splitting plane, f_{ct} is the concrete shear stress limit at potential vertical plane. Park and Paulay⁽⁴⁷⁾ proposed a simple expression for f_{ct}

$$f_{ct} = 0.166 \sqrt{f'_c} \quad (5.11)$$

A_{sp} is calculated as:

$$A_{sp} = (L-h)b \quad (5.12)$$

Eq. (5.10) is then expressed as:

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$$S_2 = 0.166k_2(L-h)b\sqrt{f'_c} \quad (5.13)$$

Many researchers, including Schlaich et al. (1987)⁽⁴⁸⁾ and Schlaich and Schaefer (1991),⁽⁴⁹⁾ suggest that the compression stress in compression strut is limited to $0.4f'_c$, then the compression force C_c can be expressed as:

$$C_c = 0.4k_3bhf'_c/\sqrt{2} = 0.283k_3f'_cA_g \quad (5.14)$$

where k_3 is a factor expressing the level of compression stress in concrete strut $k_3 \leq 1$.

From Eqs. (5.9), (5.13) and (5.14), Eq. (5.8) is then expressed as:

$$V = k_1 \frac{L-h}{s} A_v f_{yt} - 0.166k_2(L-h)b\sqrt{f'_c} + (n - 0.8k_3)f'_c A_g \quad (5.15)$$

Parametric studies with three variables, k_1 , k_2 , k_3 , showed that for columns failed by Mode 1 (splitting failure), the best match for V_{max} was achieved when k_2 approach 1.

k_1 is found to be a function of transverse reinforcement ratio (ρ_v %) expressing the level of stress in transverse reinforcement at maximum shear force, proportional with the ratio of transverse reinforcement:

$$k_1 = 0.9e^{-0.9\rho_v} \quad (5.16)$$

k_3 is a function of the applied axial force showing the level of axial stress transferred through the compression zone (C_1) depending on the level of applied axial force ($n = P/f'_c A_g$) as shown in **Figure 5.8**:

$$k_3 = 1.17n - 0.1 \quad (5.17)$$

Figure 5.8 Curves fitting of parameter k_1 , k_3

Equation (5.15) can then be rewritten as:

$$V = 0.9 \frac{L-h}{s} A_v f_{yt} e^{-0.9\rho_v} - 0.166(L-h)b\sqrt{f'_c} + (0.08 + 0.064n)f'_c A_g \quad (5.18)$$

Verification of the proposed equation (Equation 5.18, $k_2=1$) using the test results from 17 RC columns with splitting cracks is shown in **Table 5.1**. The proposed equation yields good predicted shear strength with the mean difference between the analytical assessments and the test results of 5% and the coefficient of variation (CV) of 14.9%. This pointed out that when the specimens reached maximum shear force, the splitting stress also reached the splitting ultimate strength while the stresses in other force transferring mechanisms (C_c and T_s) were still in elastic range.

Table 5.1 Shear strength calculated by the proposed model for columns with observed splitting cracks

Author	Specimen	L (mm)	b (mm)	h (mm)	f _c (Mpa)	f _{st} (Mpa)	d (mm)	A _v (mm ²)	s (mm)	ρ _s (%)	αd	n= P/f _c A _g	V _{test}	V _{pro}	V _{test} / V _{pro}
	SP1-1.7-0.2	1200	350	350	29.8	475	300	56.5	125	0.129	1.71	0.20	238.3	208.9	1.14
	SP2-1.7-0.35	1200	350	350	29.2	475	300	56.5	125	0.129	1.71	0.35	263.2	239.3	1.10
	SP3-2.4-0.2	1700	350	350	30.6	475	300	56.5	125	0.129	2.43	0.20	193.3	135.8	1.42
	SP4-2.4-0.35	1700	350	350	28.7	475	300	56.5	125	0.129	2.43	0.35	198.0	162.0	1.22
Current test	SR1-1.7-0.35	1700	250	490	23.3	509	440	56.5	125	0.092	1.73	0.35	263.0	274.7	0.96
	SR2-1.7-0.5	1700	250	490	22.5	509	440	56.5	125	0.092	1.73	0.50	278.0	295.4	0.94
	SW1-1.4-0.2	1700	200	610	21.7	509	560	56.5	125	0.074	1.39	0.20	247.8	284.2	0.87
	SW2-1.4-0.35	1700	200	610	20.2	509	560	56.5	125	0.074	1.39	0.35	266.0	297.0	0.90
Tran (2010) ⁽¹⁹⁾	SW3-1.4-0.5	1700	200	610	20.5	509	560	56.5	125	0.074	1.39	0.50	289.0	323.5	0.89
	RC-1.7-0.05	1700	250	490	32.5	392.6	440	56.5	125	0.092	1.73	0.05	283.1	216.0	1.31
Marina& Theodosio (2006)	RC-1.7-0.5	1700	250	490	26.8	392.6	440	56.5	125	0.092	1.73	0.50	355.2	279.4	1.27
	specimen 7	1000	250	250	38	300	210	150.8	50	1.206	2.00	0.30	210.0	245.3	0.86
Harumi Yashiro et al (1988) ⁽⁵⁰⁾	15-0.56@90-6B	750	250	250	28.84	378	200	127.2	90	0.565	1.50	0.08	158.0	183.9	0.86
	15-0.85@60-6B	750	250	250	38.03	378	200	127.2	60	0.848	1.50	0.06	198.0	236.3	0.84
	15-1.28@60-6B	750	250	250	31.4	378	200	190.8	60	1.272	1.50	0.08	185.0	220.1	0.84
	30-0.56@90-6B	750	250	250	29.23	378	200	127.2	90	0.565	1.50	0.16	250.0	194.5	1.29
	45-0.56@90-6B	750	250	250	29.23	378	200	127.2	90	0.565	1.50	0.26	240.0	206.2	1.16
														Ave	1.05
														CV	0.149

5.4 Proposal of a New Shear Strength Model

To extend the application of the proposed shear strength model to RC columns in which there is a possibility of splitting failure to occur, Equation 5.16 can be further modified by verifying the parameter k_2 as a function of shear span to depth ratio (a/d) and applied axial force ($n=P/f'_c A_g$). k_2 is obtained based on test data of the above 17 columns:

$$k_2 = n^{0.1} + 0.2 \left(\frac{a}{d} \right)^{-2} \quad (5.19)$$

Equation (5.15) can be rewritten as:

$$V = 0.9 \frac{L-h}{s} A_v f_{yt} e^{-0.9 \rho_v} - 0.166 \left[n^{0.1} + 0.2 \left(\frac{a}{d} \right)^{-2} \right] (L-h) b \sqrt{f'_c} + (0.08 + 0.064n) f'_c A_g \quad (5.20)$$

The proposed equation (5.20) can be verified by using the maximum shear forces recorded in the published 89 column tests that are available to the author. These specimens were selected mainly on the basis of the observed failure mode whereas there is a possibility that the splitting failure might occur, which are: short columns (shear span to depth ratio smaller than 2.5) and tested under double curvature cyclic bending. The selected specimens had the sizes ($b \times h \times L$) ranging from 80x80x160 to 400x500x1650 mm. Concrete compressive strength ranged from 15.1 to 48.0 MPa, whereas the transverse reinforcement ratio ranged from 0.1 to 1.88%. The applied axial force ratio ($P/f'_c A_g$) ranged from 0.0 to 0.9. It is noted that of the 89 specimens, there are 17 specimens which observed splitting failure. Details of the selected specimens and their shear strength calculated based on the proposed equation are tabulated in **Table 5.3**.

Shear strength were also estimated based on the methods from ACI 318-11, ASCE 41 and Biskinis et al. (2004) and compared with the proposed model. **Figure 5.9** plots the comparisons of the measured shear strength with the calculated shear strengths $V_{proposed}$, V_{ACI} , V_{ASCE} and $V_{biskinis}$. The mean ratio of the measured to calculated shear strengths and its coefficient of variation are 1.03 and 0.28, 1.20 and

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0.31, 1.05 and 0.39 and 1.13 and 0.36 for the proposed model and models from ACI 318-08, ASCE 41 and Biskinis et al., respectively. It is noted that although the proposed model and ACI model are independent on displacement ductility, ACI model provides reasonably conservative estimates for shear strength of the collected columns. The shear strength model in ASCE 41, originally developed for columns with light reinforcement (Sezen and Moehle, 2004), is sensitive to displacement ductility demand. Even after considering the ductility ratio from experimental data, the shear strengths estimated by ASCE 41 are inconsistent and room for improvement remains. On the contrary, the purely empirical shear strength model for columns controlled by diagonal compression (Equation 2.6a) of Biskinis et al. relies on little variation of the displacement ductility factor. The Biskinis et al. model produces a large scatter for shear strength predictions of short columns with shear span to depth ratio equal or less than 2.5 in this study. This might be because the shear strength model for columns controlled by diagonal compression was built by fitting parameters to test results of columns with shear span to depth ratio from 1 to 2. Among the four shear strength models, the proposed model gives the best match with the experimental results of the database.

To investigate the validity and applicability of the proposed equation across the range of several key parameters including axial load, shear span to depth ratio, compressive strength of concrete and transverse reinforcement ratio, the ratio of experimental shear strength (V_u) to shear strength calculated from the proposed Eq. (5.21) against axial load ($P/f'_c A_g$), shear span to depth ratio (a/d), transverse reinforcement index-to-concrete grade ratio ($\rho_v f_{yt}/f'_c$) is plotted in **Figure 5.10**. The good correlation between the experimental and predicted strengths across the range of axial load, shear span to depth ratio, transverse reinforcement index indicates that the proposed model well represents the effects of these key parameters. It can therefore be concluded that the proposed shear strength model is applicable and reliable. Nevertheless, it should be noted, the proposed model under estimate the shear strength for column with a/d equal to 1 while overestimate the shear strength for column with a/d equal to 2.5.

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Figure 5.9 Calculated shear strength for different models

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Figure 5.10 Variation of experimental to predicted strength ratio as a function of key parameters

Table 5.2 Comparisons of shear strength calculated by proposed model and existing models for short columns

Authors	Specimens	L	b	h	f _c	f _y	A _v		s	ρ _v (%)	a/d	n= P/f _c A _g	kN			
							mm ²	mm					V _{test}	V _{pro}	V _{ACI}	V _{Biskinis}
Current tests	SP1-1.7-0.2	1200	350	350	29.8	475	56.5	125	0.129	1.71	0.20	238.3	249.4	201.2	303.1	337.8
	SP2-1.7-0.35	1200	350	350	29.2	475	56.5	125	0.129	1.71	0.35	263.2	260.5	229.0	354.0	433.8
	SP3-2.4-0.2	1700	350	350	30.6	475	56.5	125	0.129	2.43	0.20	193.3	216.1	204.1	235.9	278.3
	SP4-2.4-0.35	1700	350	350	28.7	475	56.5	125	0.129	2.43	0.35	198.0	210.1	226.4	266.4	312.5
	SR1-1.7-0.35	1700	250	490	23.3	509	56.5	125	0.092	1.73	0.35	263.0	294.4	242.0	357.5	365.0
	SR2-1.7-0.5	1700	250	490	22.5	509	56.5	125	0.092	1.73	0.50	278.0	303.3	259.1	389.6	435.7
	SW1-1.4-0.2	1700	200	610	21.7	509	56.5	125	0.074	1.39	0.20	247.8	303.5	243.1	382.3	297.5
	SW2-1.4-0.35	1700	200	610	20.2	509	56.5	125	0.074	1.39	0.35	266.0	304.1	255.7	423.1	372.6
	SW3-1.4-0.5	1700	200	610	20.5	509	56.5	125	0.074	1.39	0.50	289.0	322.8	276.1	471.1	466.4
	SC-2.4-0.2	1700	350	350	22.6	392.6	56.5	125	0.129	2.43	0.20	218.9	181.6	171.8	196.0	233.0
Tran 2010 ⁽¹⁰⁾	SC-2.4-0.35	1700	350	350	24.2	392.6	56.5	125	0.129	2.43	0.35	237.6	219.0	201.7	235.3	303.0
	SC-1.7-0.05	1200	350	350	29.8	392.6	56.5	125	0.129	1.71	0.05	276.4	229.7	167.0	222.2	229.7
	SC-1.7-0.2	1200	350	350	27.5	392.6	56.5	125	0.129	1.71	0.20	294.2	221.3	190.6	282.0	319.7
	SC-1.7-0.35	1200	350	350	25.5	392.6	56.5	125	0.129	1.71	0.35	335.5	236.5	208.7	319.4	396.3
	SC-1.7-0.5	1200	350	350	26.4	392.6	56.5	125	0.129	1.71	0.50	375.6	249.2	240.6	367.8	496.2
	RC-1.7-0.05	1700	250	490	32.5	392.6	56.5	125	0.092	1.73	0.05	283.1	201.1	201.5	262.0	228.3

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Authors	Specimens	L	b	h	f _c	f _y	A _v		s	ρ _v (%)	a/d	n= P/f _c A _g	V _{test}	V _{pro}	V _{ACI}	V _{ASR/E}	V _{Riskins}
							mm	mm ²									
Amitasu et al. 1991 ⁽⁵⁵⁾	CB060C	646	278	278	46.3	414	113.0	52	0.782	1.16	0.74	509.0	471.6	485.3	548.1	636.1	
	15-0.56@90-6B	750	250	250	28.8	378	127.2	90	0.565	1.50	0.08	158.0	209.6	159.4	205.4	117.4	
	15-0.85@60-6B	750	250	250	38.0	378	127.2	60	0.848	1.50	0.06	198.0	269.6	220.6	270.0	131.5	
Harumi et al. 1990 ⁽⁵⁶⁾	15-1.28@60-6B	750	250	250	31.4	378	190.8	60	1.272	1.50	0.08	185.0	247.2	295.3	342.3	121.5	
	30-0.56@90-6B	750	250	250	29.2	378	127.2	90	0.565	1.50	0.16	250.0	212.1	167.6	227.0	129.8	
	45-0.56@90-6B	750	250	250	29.2	378	127.2	90	0.565	1.50	0.25	240.0	216.2	175.4	244.8	141.6	
Arakawa et al. 1989 ⁽⁵⁶⁾	OA2	450	180	180	31.8	249	25.1	64.3	0.217	1.25	0.18	124.3	71.9	55.4	69.3	116.6	
	OA5	450	180	180	33.0	249	25.1	64.3	0.217	1.25	0.45	120.7	87.3	74.1	94.2	185.9	
Ono et al. 1989 ⁽⁵⁷⁾	CA025C	600	200	200	25.8	426	113.0	70	0.807	1.50	0.26	130.0	156.8	165.4	152.8	118.7	
	CA060C	600	200	200	25.8	426	113.0	70	0.807	1.50	0.62	135.2	172.6	185.4	177.7	161.4	
Ikeda et al. 1986 ⁽⁵⁸⁾	43	1000	200	200	19.6	434	56.0	100	0.280	2.50	0.10	74.0	114.6	71.2	64.1	53.0	
	44	1000	200	200	19.6	434	56.0	100	0.280	2.50	0.10	77.0	114.6	71.2	71.2	54.7	
	45	1000	200	200	19.6	434	56.0	100	0.280	2.50	0.20	82.0	110.3	74.8	77.4	62.3	
	46	1000	200	200	19.6	434	56.0	100	0.280	2.50	0.20	81.0	110.3	74.8	77.4	62.9	

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Authors	Specimens	L	b	h	f _c	f _y	A _v	s	ρ _v (%)	a/d	n= P/f _c A _g	kN					V _{BSKINS}
												V _{test}	V _{pro}	V _{ACI}	V _{ASCE}	V _{BSKINS}	
Woodward and Jirsa 1984 ⁽⁶⁴⁾	2CUS	910	410	230	42.0	414	113.0	89	0.552	1.98	0.27	375.5	312.9	500.1	402.5	575.3	
	C-86-14	910	300	300	34.8	414	56.5	65	0.290	1.52	0.20	267.0	282.9	167.5	152.3	141.2	
	C-86-09	910	300	300	32.1	414	56.5	101	0.186	1.52	0.22	289.3	223.0	144.5	131.7	137.9	
	C-86-03	910	300	300	42.1	414	56.5	303	0.062	1.52	0.14	324.9	209.3	127.0	110.5	142.0	
	C-84-32	910	300	300	33.5	414	56.5	28	0.662	1.52	0.21	280.4	371.9	231.1	217.1	139.6	
Imai and Yamamoto 1983 ⁽⁶⁵⁾	C-84-21	910	300	300	33.5	414	56.5	43	0.435	1.52	0.21	275.9	323.5	190.8	176.8	132.3	
	C-84-14	910	300	300	37.6	414	56.5	65	0.290	1.52	0.18	271.5	298.3	172.0	154.4	136.9	
Arakawa et al. 1982 ⁽⁶⁶⁾	No. 1	1650	400	500	27.1	336	127.2	100	0.254	1.65	0.07	442.9	506.0	372.2	493.9	438.7	
	No. 102	750	250	250	20.6	323	71.2	32	0.890	1.50	0.33	159.7	185.3	182.0	252.0	98.9	
Nagasaka et al. 1982 ⁽⁶⁷⁾	HPRC10-63	600	200	200	21.6	344	47.5	35	0.678	1.50	0.17	87.2	115.7	120.4	144.5	82.7	
	HPRC19-32	600	200	200	21.0	344	47.5	20	1.187	1.50	0.35	111.7	128.2	190.9	223.6	97.7	
Ramirez and Jirsa 1980 ⁽⁶⁸⁾	00-U	915	305	305	34.5	455	63.3	65.3	0.318	1.50	0.00	265.0	422.2	199.7	241.0	210.6	
	120C-U	915	305	305	30.7	455	63.3	65.3	0.318	1.50	0.19	290.0	292.7	226.8	320.5	322.8	
Ave (V _{Test} /V _f)												1.03	1.20	1.05	1.13		
CV (V _{Test} /V _f)												0.28	0.31	0.39	0.36		

5.5 Effects of Column Parameters

In this section, a parametric study is conducted to improve the understanding of the effects of various parameters on the shear strength of RC columns based on the proposed model. The parameters investigated were the shear span to depth ratio (a/d), column axial load ratio ($P/f'_c A_g$), concrete compressive strength (f'_c) and transverse reinforcement index ($\rho_v f_{yt}$). Specimen SP1 with a shear span to depth ratio of 1.7 was considered as the reference specimen in the parametric study.

5.5.1 Influence of Shear Span to Depth Ratio

Figure 5.11 shows the influence of the shear span to depth ratio on shear strength of RC columns. Six levels of shear span to depth ratios, $a/d = 1.0, 1.2, 1.50, 1.8, 2.10$, and 2.43 , were investigated. In general, the shear strength decreased with an increase in shear span to depth ratio. It was found that when the shear span to depth ratio was increased from 1.00 to 1.20, 1.50, 1.80, 2.10 and 2.43, the shear strength of columns axial loads of $0.02f'_c A_g$ dropped by approximately 12.8, 19.4, 28.2, 38.5 and 54.7%, respectively. Similar trends were observed for the columns with other levels of axial load ratio. The shear strength increased by approximately 6.9, 16.0, 26.7, 39.6 and 56.9% for columns under an axial load of $0.60f'_c A_g$ with the shear span to depth ratio increasing from 1.00 to 1.20, 1.50, 1.80, 2.10 and 2.43, respectively. This confirmed that the shear span to depth ratio significantly influences the shear strength of RC columns.

5.5.2 Influence of Axial Force

It is generally recognized that the presence of column axial load can effectively increase the shear strength of columns. The analyses illustrated in **Figure 5.11** were carried out to assess the influence of axial load ratio on shear strength. The axial load ratio was varied from 0 to 0.60. In general, the shear strength increased with an increase in axial load ratio. It is shown that with an increase in axial load ratio from 0.1 to 0.30, 0.40 and 0.60 the strength for specimens with a shear span to depth ratio of 1.0 rose by approximately 10.3, 16.8 and 30.5%, respectively. The trends gradually slowed down with the increase in shear span to depth ratio. This is

understandable because when the height of a column is increased (the section is kept unchanged) the capacity of diagonal compression strut is reduced, then the applied axial force cannot fully affect the shear carrying mechanism. Hence, it can be concluded that when the shear span to depth ratio of a column is decreased, the axial load ratio affects more significantly to the shear strength of the column.

Figure 5.11 Effects of key variables on shear strength

5.5.3 Influence of Transverse Reinforcement

The analyses illustrated in **Figure 5.11** were conducted to assess the influence of transverse reinforcement on shear strength of short columns. Two axial loads of $0.2 f'_c A_g$ and $0.35 f'_c A_g$ were considered. Since the results for these two cases were similar, only one case was demonstrated here. Transverse reinforcement ratio was varied by changing the cross sectional area or spacing of tie bars. In this study, the effect of transverse reinforcement was expressed by the transverse reinforcement index which was taken as multiplying transverse reinforcement ratio (ρ_v) by its yield strength (f_{yt}). It was found that beyond a particular ratio, the shear strength

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remained almost the same regardless of changing the spacing or cross sectional area of transverse reinforcement. **Figure 5.11** shows that when transverse reinforcement index increased from 0.36 to 0.9, 1.45, 1.99 and 2.53 MPa, shear strength rose significantly by approximately 95.8, 151, 178, 189%, respectively for columns under an axial load of $0.20 f'_c A_g$. This trend is similar for columns having different concrete compressive strength. The shear strength was almost unchanged when the transverse reinforcement index was increased beyond 3.1 MPa. This suggested that the effect of transverse reinforcement index on shear strength is significant up to a certain ratio. It is worth noting that the contribution of transverse reinforcement on shear strength in most of current approaches is based on the total shear force carried by transverse reinforcement at the diagonal cracking plane. Therefore, shear strength is simply proportional to the transverse reinforcement ratio without any limitation. In other words, shear strength will keep increasing as long as the transverse reinforcement index is increased. However, for high amounts of transverse reinforcement the concrete in the inclined struts may fail before yielding of stirrup. Then the shear strength will not be governed by transverse reinforcement any longer. Therefore, certainly, there is a limit on shear strength of RC columns when its transverse reinforcement index is increased.

5.6 Summary

1. Splitting failure of RC columns was the result of the development of vertical cracks along the mid-height at the central region of columns, and it was found only take place in columns with a shear span to depth ratio not higher than 2.5. This phenomenon appeared in both columns with deformed rebars and plain rebars, and was not directly related to the level of applied axial compression force. Columns that failed by splitting failure reached a lower shear capacity but a higher drift capacity compared to those failed by diagonal shear failure.
2. An alternative load-transfer path was proposed to explain that splitting failure was due to the splitting shear stress exceeding the splitting ultimate strength along vertical potential cracks. A new shear strength model was then formulated based on the proposed failure mechanism. Comparisons made between the shear strength of analytical results (Equation 5.19) and test data collected on 17 columns observed

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splitting cracks showed a good agreement with an average difference of 5% and coefficient of variation of 0.135. The model also gave good correlations with observed behavior.

3. The extended model could be used to estimate the shear strength of short columns. The proposed shear strength model (Equation 5.21) was verified with the experimental data collected from 89 RC columns, having shear span to depth ratio not higher than 2.5, showed a better agreement than the models of ACI 318 (2011), ASCE 41 (2007) and Biskinis et al. (2004). The proposed model suggests that, for short columns, the transverse reinforcement index ($\rho_v f_{yt}$) could be used effectively (for improving shear strength) and economically up to 3.1MPa.

CHAPTER VI: EFFECTS OF SEISMIC LOADING DIRECTIONS ON SEISMIC RESPONSE OF RC COLUMNS

6.1 Introduction

There have been extensive experimental studies conducted on RC column subjected to seismic loading, most of them focused on the principal loading direction. Thus far, few research works have been done on non-principal loading. However, real structures behave multi-directionally when subjected to earthquake excitation; therefore, it is essential to evaluate the effect of multidirectional loading on the seismic performance of reinforced concrete columns in order to develop more reliable design and assessment procedures. This chapter aims to provide better understanding on the seismic behavior of existing RC columns under multi-directional loadings. Seven half-scale existing RC columns with deformed rebars and light transverse reinforcement were tested under a combination of a constant axial load and quasi-static cyclic loadings simulating earthquake actions. Details of test specimens, namely Series 2, have been presented in Chapter 3; a schematic diagram of the loading apparatus is shown in **Figure 6.1**. The experimental results, including crack patterns, hysteretic response, shear strength, shear failure, axial failure and cumulative energy dissipation, were compared with the previous test results of similar columns but loaded along principal directions and with results obtained from a nonlinear finite element analysis. The findings are useful in understanding the response and failure mechanisms of existing RC columns under different loading directions. Understanding their deficiencies would make it possible to improve the survivability of such columns in the event of earthquakes.

Figure 6.1 Experimental setup

6.2 Experimental Results and Discussions

This section presents the results obtained from the tests of Series 2 comprising of seven specimens. Results of the tests on columns under principal loading direction from Tran (2010)⁽¹⁹⁾ are also added for comparisons. Three specimens in Tran's tests, namely SC-2.4-0.20, SC-1.7-0.2 and RC-1.7-0.35, are selected and renamed as S2.4-0, S1.7-0 and R1.7-0, respectively. Details of these specimens have been presented in Chapter 2. It is noted that the previous tests and the current tests were conducted using the same test setup; the only different parameter was the loading direction of the horizontal load regarding to the principal axis of column sections. More details of experimental results for each specimen can be found in Appendix A3.

6.2.1 Crack Patterns and Failure Modes

The crack patterns of the test specimens at shear failure and axial failure are shown in **Figure 6.2**. Shear failure was recorded when the columns lost over 20% of lateral resistance while axial failure happened once the specimens were no longer able to resist the applied axial force. The important features in the crack development are highlighted.

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Shear Failure	Axial Failure	Shear Failure	Axial Failure	Shear Failure	Axial Failure		
S2.4-0		S2.4-30		S2.4-45

Shear Failure	Axial Failure	Shear Failure	Axial Failure	Shear Failure	Axial Failure		
S1.7-0		S1.7-30		S1.7-45

Shear Failure	Axial Failure	Shear Failure	Axial Failure	Shear Failure	Axial Failure	Shear Failure	Axial Failure
R1.7-0		R1.7-30		R1.7-45		R1.7-60	

Figure 6.2 Failure modes of test specimens

(Specimens S2.4-0, S1.7-0 and R1.7-0 were tested in principal direction by Tran⁽¹⁹⁾)

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All the tested specimens developed fine flexural cracks at both ends of the columns when loaded up to a drift ratio of 0.4%. In subsequent cycles, diagonal cracks were observed, as the flexural cracks extended from the two edges in the loading direction of the columns to the middle area, before the columns reached maximum shear force. In the group S2.4 (specimens S2.4-30 and S2.4-45), the first flexural cracks were observed at a drift ratio of 0.25%, regardless of loading angles. These columns also experienced the first diagonal crack at the same drift ratio of 1.25%. A similar observation was recorded at another square-section group, S1.7 (specimens S1.7-30, S1.7-45); those first flexural and diagonal cracks appeared at a drift ratio of 0.2% and 0.4%, respectively. However, in the rectangular group, R1.7 (specimens R1.7-30, R1.7-45 and R1.7-60), there were substantial changes in the drift ratio corresponding to those two points upon changing the loading angles. Vertical cracks developed throughout the length of the longitudinal rebars when loaded in non-principal directions while the columns tested in the principal direction only developed inclined cracks at the end of the test.

There were two main axial failure modes. In the first mode, as observed in group S2.4 and two specimens S1.7-0, R1.7-0, the steep shear crack, which had developed in the column from the previous stages, became wider. The column height was then divided into two parts by this shear crack. There was sliding between the two parts in the crack surfaces, leading to buckling of longitudinal reinforcing bars as well as fracturing of transverse reinforcing bars along this shear crack. The axial failure occurring in those specimens was believed to be due to the shear friction demand exceeding the shear friction capacity. In the second mode, as found in S1.7-30, S1.7-45; R1.7-30, R1.7-45 and R1.7-60, vertical cracks appeared from the early cycles and continued developing without distinct diagonal cracks. This was due to the sliding in the surface of longitudinal reinforcement and surrounding concrete, when the bond stress exceeds the splitting strength as studied in Chapter 5. The

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cross-section of the specimens was then gradually reduced, leading to shear failure and axial failure. It was observed in group S2.4 that the axial failure mode did not change when the loading direction of lateral force was rotated; the only noticeable observation was the angle of the diagonal cracks, which was changed from 60° (with respect to the column axis) in specimens S2.4-0 and S2.4-30, to 45° in specimen S2.4-45. However, the axial failure modes were changed from mode 1 to mode 2 on both other groups, S1.7 and R1.7, when loading out of the principal directions.

6.2.2 Hysteretic Responses

The hysteretic responses of the test specimens with the theoretical flexural strength and nominal shear strength are shown in **Figure 6.3**. Comparisons of envelope responses are shown in **Figure 6.4**. Overall, the responses were more ductile when the angle of loading direction was changed from 0° to 45° . Specimens with brittle response, S1.7-30, S1.7-45 and R1.7-30, did not reach the nominal shear strength. The hysteretic loops of all test specimens showed the degradation of stiffness and load-carrying capacity during repeated cycles due to the cracking of the concrete and yielding of the reinforcing bars, together with that due to the second order bending moment resulting from the applied axial load. The pinching effect was observed in the hysteretic loops of all the test specimens. Detail hysteretic response of the test specimens will be assessed based on the initial stiffness, shear strength, drift ratio at axial failure and cumulative energy dissipation in the following sections.

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Figure 6.3 Hysteretic responses of the specimens (Series 2)

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Figure 6.4 Comparisons of backbone curves

6.2.3 Initial Stiffness

The definition of initial stiffness has been presented in Chapter 4 (see Section 4.3.3). The initial stiffness of all the test specimens is illustrated with respect to its loading direction in **Figure 6.5**. It is shown that the initial stiffness of group S1.7 was almost unchanged at around 25 kN/mm where the highest value was reached at a loading angle of 30°. The similar trend was observed in the other square columns, S2.4, with slightly smaller stiffness of 10.2 and 11.8 kN/mm when loaded from the principal and 45° direction, respectively. Overall, initial stiffness was observed to increase by 100% when the shear span to depth ratio (in the principal direction) was decreased from 2.4 to 1.7 (under applied axial forces of $0.2f_cA_g$). In the rectangular group, R1.7, the highest initial stiffness also resulted from the lateral force angle of 30° to the strong direction of the column. This value decreased by approximately 20% when the loading angle increased to 45° and 60°.

Figure 6.5 Initial stiffness versus loading directions

6.2.4 Shear Strengths

Shear strengths of the test specimens are shown in **Table 6.1**. The shear strength of group S2.4 was achieved when the specimens were loaded up to a drift ratio of

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around 1.4%. Similar observations were also recorded in the other specimens group, S1.7, where shear forces reached the maximum at drift ratio of 1.05% and 0.97% in S1.7-45 and S1.7-30, respectively. It is noted that the two square groups were subjected to the same level of axial force of $0.2f_c A_g$. It can be concluded that the shorter columns reach the maximum shear force at lower drift ratios. For the third group, R1.7, the shear strengths of three rectangular columns were also attained at close values of the drift ratio. They were 1.21%, 1.25% and 1.26% when the loading angles of shear forces were 30° , 45° , 60° to the strong direction of the columns, respectively. The data, together with the above discussion, clearly indicates that seismic shear strength of a column will be reached a particular drift ratio regardless of loading directions.

Figure 6.6 shows the relationship between shear strength of columns and direction of horizontal forces from both experimental and predicted results. In this figure the lines connecting two shear strength values in principal directions (taken from previous test results) were plotted as the equation of a circle with the center located at the center point of columns and radius as the shear strength in principal directions. Only a 5% difference for group S1.7 was observed, while there was almost no change in group S2.4. This provides evidence that for square sections, columns in non-principal directions will reach the same value of maximum shear force regardless of loading directions.

For the case of columns with rectangular section, R1.7, it is obvious that, with an increase in loading angle from the strong direction to the weak direction, the shear strength of columns will decrease. The recorded data was 253.6 kN, 221.1 kN and 190.0 kN for loading angles 30° , 45° and 60° , respectively. However, it is found from curves fitting approach that those values can be linked by the ellipse equation as follows

$$\frac{V_{ax}^2}{V_x^2} + \frac{V_{ay}^2}{V_y^2} = 1 \quad (6.1)$$

where V_x and V_y are the shear strengths in the strong and weak directions, respectively; V_{ax} and V_{ay} are the respective components of strength in the diagonal direction when projecting to the strong and weak directions; The shear strength in the diagonal direction is calculated as $\sqrt{V_{ax}^2 + V_{ay}^2}$. In other words, shear strength of rectangular columns in non-principal directions can be predicted based on the shear strength in principal directions. This is similar to the finding on short columns by Umehara and Jirsa.⁽³³⁾ It is noted that the shear strength predicted by combining the approaches specified in ASCE 41 and the one proposed by Umehara and Jirsa is found to be overestimated in all specimens.

Figure 6.6 Shear strength (kN) versus loading directions (degree)

6.2.5 Drift Ratios at Axial Failure

Axial failure occurs when columns are unable to resist a certain level of applied axial force ($0.2f'_cA_g$ for square groups and $0.35f'_cA_g$ for rectangular groups); the drift ratios at axial failure of the test specimens are illustrated in Figure 6.7. Generally, an increase in the angle of the loading direction corresponds to a growth

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in the drift ratio at axial failure. There was an increase of about 2.15% when the angles changed from 0 to 30° at all square columns. Group S2.4 showed the increase from 2.82% to 4.96% while in group S1.7 the change was to 4.0% from 1.82%. These two groups reached maximum drift ratio at axial failure while being at 45°. The group of shorter columns, S1.7, continued along the same gradient when loading from 0° to 45° to 60° while the longer group, S2.4, increased at a smaller rate to reach 5.41% at 45°. An analogous trend was also observed in the group of rectangular columns, R1.7, whose drift ratios experienced an increase of around 26% and 34%, when rotating the loading angle from the strong direction to 30° and 45°. This group was found to produce the largest drift ratio when loaded at 60°.

It should be noted that most of the existing models^{(17),(69)} for predicting ultimate drift ratio are based on an assumption of a shear-plane failure in which the equilibrium of component forces is considered on a shear-friction plane. Axial failure is defined when the shear friction demand exceeds the shear friction resistance along this plane. The above methods seem unsuitable for application on columns failing in the second mode, without forming the shear failure plane.

Figure 6.7 Drift ratio at axial failure versus loading directions

6.2.6 Cumulative Energy Dissipation

The cumulative energy dissipation at each performance level of all specimens obtained from the test is shown in **Table 6.1**. It can be seen, most remarkably, that an increase in the maximum cumulative energy dissipation (at PL4) was recorded for both groups S2.4 and S1.7 when loading direction was changed from 0° to 45°. The results also show that the maximum cumulative energy dissipations of group S1.7 were almost 60% lower than those of the group S2.4 for the same loading directions. This is understandable since RC columns with a smaller shear span to depth ratio have more brittle response. An analogous trend was observed for the rectangular group in which the maximum cumulative energy dissipations gradually increased with an increment of around 2 kNm when the loading directions were changed from 0°, 30°, and 45° to 60°. It can therefore be concluded that the maximum cumulative energy dissipations of the columns had a close relationship to the drift ratio at axial failure.

Table 6.1 Cumulative Energy Dissipation at each performance level of specimens

(Refer to Section 4.2 for the definitions of the 4 PLs)

Specimens	Cumulative Energy Dissipation at (kNm)			
	PL1	PL2	PL3	PL4
S2.4-0	8.5	12.5	22.8	34.9
S2.4-30	2.5	7.2	17.2	38.2
S2.4-45	1.2	6.7	18.7	42.7
S1.7-0	-	7.2	8.5	13.5
S1.7-30	-	4.2	5.8	17.2
S1.7-45	2.3	3.4	6.4	18.6
R1.7-0	6.5	12.5	23.0	26.5
R1.7-30	-	9.6	16.6	28.2
R1.7-45	4.7	8.8	23.4	30.6
R1.7-60	2.7	7.3	26.1	33.4

6.2.7 Strains in Reinforcement

Figure 6.8 Local strain at longitudinal reinforcement (a) and transverse reinforcement (b) of specimen S2.4-45

The strain gauge readings were found to agree well with the bending moment patterns in all specimens before the columns failed in shear. Details of strain gauge measurement for each specimen can be found in Appendix A3. Typical results are presented herein. As shown in **Figure 6.8**, the largest recorded tensile strain of 2670 μ was observed at location L3e in specimen S2.4-45, as a typical illustration, which was 250mm from the column-top base interface and at the corner bar in the loading

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direction plane. It can be seen from **Figure 6.8a** that the corner bar reached yield strain just after the specimen reached maximum shear force at a drift ratio of 1.42%.

The measured strains in the transverse reinforcing bars of specimen S2.4-45 are illustrated in **Figure 6.8b**. Up to a drift ratio of 2, the measured strain did not increase significantly with an increase of the drift ratios. No yielding of transverse reinforcement was observed prior to the occurrence of shear failure. However, in the subsequent cycles, the strain at position V6 had a sudden spike and reached the fracture strain. Compared to the crack patterns, the strain gauge reading matched the stress distribution on the specimens well.

6.3 Finite Element Analysis

The complex behavior of RC columns under seismic loadings requires further analysis to supplement the experimental program. A numerical study, consisting of three-dimensional (3D) nonlinear Finite Element (FE) models, was therefore carried out. The analysis was conducted by using Engineer's Studio program with fiber element approach.

6.3.1 Description of Fiber Model

The fiber element approach, (see, e.g. Taucer et al. (1991)⁽⁷⁰⁾), has been recognized as one of the most promising methods for static and dynamic analysis of beam-column elements. By using the fiber element approach, the hysteretic behavior of RC members can be captured directly from cyclic stress and strain relationships of the materials; the axial force–biaxial bending interaction can be rationally accounted for. A schematic figure from Taucer et al. to conceptually illustrate the configuration of a fiber beam–column element is illustrated in **Figure 6.9**. Each of the control cross-sections is subdivided into many filaments of concrete fibers and steel reinforcing fibers. There are a number of control cross-sections along the

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element. These sections are located at the control points of the numerical integration scheme. The characteristics of the element, such as element flexibility and element displacements, are obtained from integrating their sectional components along the element. The corresponding sectional components, such as section flexibility and section forces, are calculated from summing the contributions of all the fibers on the specific cross-section. The section force and deformation relation of RC fiber section, the fiber element formulation, and the analysis procedures of the fiber model can be found in the project report by Taucer et al., (1991).⁽⁷⁰⁾

Figure 6.9 Illustration of Fiber Beam-Column Element (Taucer et al. 1991)⁽⁷⁰⁾

6.3.2 Fiber Model for Test Specimens

As presented in previous chapters, test specimens were categorized into 3 groups, S2.4, S1.7 and R1.7. The details of each specimen given in Chapter III were used to build up the numerical model in this chapter. Only a typical specimen, S2.4-30,

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with a height of 1.7m and section 350x350mm, is depicted in this section. Other specimens were then modeled similarly.

The specimen was modeled by using three-node fiber elements with the length of 0.189m (**Figure 6.10a**). It should be noted that three - node parametrical element use the second degree curve for shape function. The influence of the shearing transformation is considered based on the Timoshenko beam theory. The curvature distribution in the longitudinal direction of the element theoretically becomes linear. The stiffness matrix of the element is constructed by the Gauss integration scheme based on the stiffness evaluation in two Gauss integration points.

The section of the specimen was modeled using three material groups: Plain concrete for the concrete cover, confined concrete for the column core; and longitudinal reinforcement. The parts of plain concrete were defined by the center line of reinforcing layers as depicted in **Figure 6.10b**.

The accuracy of the element response and, consequently, that of the entire structure are affected by the mesh selection, the number of sections used in the discrimination of a given member, and by the number of fibers in a section. A larger number of fibers certainly give better results, but computational cost increases with it, since the history variables necessary to track the hysteretic behavior of each fiber must be stored at each iteration. Therefore, a mesh convergence study was conducted and found that the best mesh was 0.01x0.01m. The section was then subdivided into 1480 filaments as shown in **Figure 6.11**.

The global axis used in the program is shown in **Figure 6.10a**. The specimen was then loaded vertically in Y axis and horizontally in X axis. The node at the bottom of the model was set to be restrained to displace in both X and Z directions without any rotation, to simulate the fixed-end boundary condition. For the top end, a rigid plate which can move in both vertical and horizontal directions was attached to

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apply vertical loads and horizontal displacements with a rotational fixity. The constant column axial load was applied as a point load. The horizontal cyclic loads were simulated by applying node displacement taken the same as the loading history presented in Chapter 3.

Figure 6.10 Typical Finite Element Model of test specimens

Figure 6.11 Typical mesh of section

6.3.3 Modeling of Concrete

To consider the effect of low confinement reinforcement ratios, the Hoshikuma et al. (1997) concrete model⁽⁷¹⁾ was used in this study. As depicted in **Figure 6.12**, the stress and strain curve of this model consists of three parts, i.e. an ascending branch, a falling branch, and a sustaining branch. The ascending branch of confined concrete reflected four boundary conditions: initial condition $f_c = 0$ at $\varepsilon_c = 0$; initial stiffness condition $df_c/d\varepsilon_c = E_c$ at $\varepsilon_c = 0$; peak condition $f_c = f_{cc}$ at $\varepsilon_c = \varepsilon_{cc}$ and E_c taken as $5000(f'_{co})^{0.5} \text{ MPa}$. The tensile strength of concrete was neglected. For unconfined concrete, the compressive strength and Young's modulus of the material tests were used. For confined concrete, compressive strength and strain at peak, f_{cc} and ε_{cc} , were calculated as follows:

$$\frac{f_{cc}}{f_{co}} = 1 + 3.8\alpha \frac{\rho_s f_{yh}}{f_{co}}; \quad \varepsilon_{cc} = 0.002 + 0.033\beta \frac{\rho_s f_{yh}}{f_{co}} \quad (6.2)$$

in which f_{co} = cylinder strength of concrete; α and β = modification factors depending on confined sectional shape: for circular $\alpha = 1.0$ and $\beta = 1.0$; for square $\alpha = 0.2$ and $\beta = 0.4$. ρ_s and f_{yh} are the ratio and yield strength of transverse reinforcement.

The falling branch of the stress-strain curve (f_c , ε_c) is approximated as a straight line and is formulated as:

$$f_c = f_{cc} - E_{des} (\varepsilon_c - \varepsilon_{cc}) \quad (6.3)$$

where, E_{des} = deterioration rate, which is developed from regression analysis of test data in the range of peak strain, ε_{cc} , to ultimate strain: ε_{cu} . The definition of ultimate strain, ε_{cu} , is when the compressive stress dropped to less than $0.5f_{cc}$. E_{des} was found experimentally as:

$$E_{des} = 11.2 \frac{f_{co}^2}{\rho_s f_{yh}} \quad (6.4)$$

The parameter K (shown in **Figure 6.12**) defines the minimum compressive stress that concrete can bear in the zone which exceeds the ultimate. The compressive stress after ϵ_{ce} becomes Kf_{cc} by using the maximum compressive stress f_{cc} . K is set to 0.2 for the confined part of the cross-section and zero for the concrete cover.

6.3.4 Modeling of Reinforcement

The stress-strain relation of steel reinforcement was modeled by a tri-linear curve as shown in **Figure 6.13**. The Menegotto-Pinto⁽⁷²⁾ model for cyclic loading was used which incorporates the Bauschinger effect for cyclic loading. For simplicity, bond-slip relationship in the reinforcement was not included in the model. Steel yield strengths were based on tensile tests of the reinforcement. A hardening ratio of 2% was assumed.

Figure 6.12 Concrete model Hoshikuma et al.(1997)⁽⁷¹⁾

Figure 6.13 Stress-strain relationship for steel reinforcement

6.3.5 Results of Finite Element Analysis

The typical comparison of results obtained from FE analysis and experiment is shown in **Figure 6.14**. Results of all other specimens are presented in **Appendix A6**. Overall, the behavioral trends obtained from finite element analysis matched

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the test results reasonably well. All specimens expressed pinching effects with good correlation as recorded from the experiment. The recorded points of interest were the maximum shear force, the drift ratio at maximum shear force, at shear failure and at axial failure which usually define the key behavior of RC columns when subjected to cyclic loading.

Figure 6.14 Typical hysteretic responses of the specimen from the test and FEA

Maximum shear forces obtained from Finite Element Analysis (FEA) in comparison with results from the experiment are summarized in **Table 6.2**. The analytical results showed a good correlation with the experimental ones in terms of shear strengths for all test specimens. It was found that the average ratio of the experimental to analytical shear strength and its coefficient of variation were 1.014 and 0.061, respectively. However, drift ratios at maximum shear force of square columns series, S2.4 and S1.7, were not captured well by FEA. The program underestimated by almost 50% the drift ratios for these columns. Better results were found at the group R1.7 especially for specimen loaded horizontally at angles of 45° or 60°. It should be noted that for rectangular series, R1.7, when the loading

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direction change from 0° - strong direction to 90° – weak direction, the specimens tended to change their behaviour from shear dominant to flexure dominant. Therefore it is evident that FEA does not provide good predictions in term of drift ratio at maximum shear force for shear critical columns. The failure of FEA in this respect, and in all others which have to do with shear-dominated phenomena, is expected; ordinary fiber models are based on the Bernoulli assumption of plane sections; neglect any nonlinearity that does not arise exclusively from normal stresses and strains and do not take into account the effects of shear stresses on the normal stress-strain behaviour.

Table 6.2 Comparisons on maximum shear forces and corresponding drift ratios

Specimen	V_{max}			Drift ratio at max. shear force		
	Exp.(kN)	FEA (kN)	Exp/FEA	Exp. (%)	FEA (%)	Exp/FEA
S2.4-30	226.4	206.7	1.10	1.40	0.7	2.00
S2.4-45	226.0	205.3	1.10	1.39	0.71	1.96
S1.7-30	268.0	277.4	0.97	0.97	0.53	1.82
S1.7-45	284.0	273.5	1.04	1.05	0.53	1.98
R1.7-30	253.6	272.4	0.93	1.25	0.85	1.47
R1.7-45	221.1	229.4	0.96	1.21	1.13	1.07
R1.7-60	190.0	190.5	1.00	1.26	1.26	1.00
Mean			1.014			1.615
CoV			0.061			0.250

Table 6.3 compares the points of shear failure and axial failure between results obtained from FEA and the experiment. It can be seen that the FEA produced good predictions in term of drift ratios at shear failure for group S2.4 and two specimens

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R1.7-30, R1.7-60, while the results for group S1.7 were overestimated. Remarkably, drift ratios at axial failure were predicted well by FEA with an average value of 0.958 and coefficient of variation of 0.088, in comparison to the experiment results. However shear resisting capacities at axial failure were not well captured by FEA.

Table 6.3 Comparisons on the points of shear failure and axial failure

Specimen	Drift ratio at Shear Failure			Shear force at Axial failure			Drift ratio at Axial failure		
	(%)			(kN)			(%)		
	Exp.	FEA	Exp/FEA	Exp.	FEA	Exp/FEA	Exp.	FEA	Exp/FEA
S2.4-30	2.35	2.06	1.14	28.9	67.9	0.43	4.96	5.00	0.99
S2.4-45	2.24	2.35	0.95	45.7	19.6	2.33	5.41	5.00	1.08
S1.7-30	1.38	2.35	0.59	49.0	115.7	0.42	4.00	5.00	0.80
S1.7-45	1.62	2.35	0.69	37.2	116	0.32	5.13	5.00	1.03
R1.7-30	1.71	1.69	1.01	54.0	81.9	0.66	2.55	2.63	0.97
R1.7-45	2.55	1.69	1.51	81.0	99	0.82	3.43	3.76	0.91
R1.7-60	2.52	2.63	0.96	42.7	57.8	0.74	3.65	3.95	0.92
Mean			0.978	0.817			0.958		
CoV			0.287	0.785			0.088		

6.3.6 Parametric Studies

Among critical parameters that influence the shear strength of RC columns, the effects of loading directions and axial force have not been well-investigated. In this section, these effects were studied and the results were compared with the test results in terms of shear strength.

6.3.6.1 Influence of Directions of Horizontal Force

The effects of loading directions were investigated in two groups - rectangular and square columns - by varying the angles of the horizontal force from 0° to 90° while all other parameters were kept constant. The results, depicted in **Figure 6.15**,

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confirmed that the shear strength of RC columns with square sections did not depend on the direction of the horizontal force. It was also found that the shear strength of rectangular columns matched well elliptical curves when the loading directions were changed. This observation was found even at different levels of axial force.

(a) Square columns (S1.7)

(b) Rectangular columns (R1.7)

Figure 6.15 Influence of loading direction and axial force to shear strength

6.3.6.2 Influence of Axial Loads

Previous experimental studies have shown that axial force is beneficial to the shear strength of RC columns. It is expressed in existing codes (ACI 318-11,⁽³¹⁾ ASCE

41⁽³⁰⁾) with linearly increasing shear strength when the applied axial force is increased, as shown in **Figure 6.16**. However, by using the Bayesian parameter estimation technique, Sasani⁽³⁹⁾ pointed out that shear strength remained at a certain value when the axial force was raised beyond the level $P/f'_c A_g = 0.42$. In this study, parametric studies were conducted to find out how axial loads at high levels influence to the shear strength.

Figure 6.16 Calculation of shear strength per existing codes

Figure 6.17 shows the numerical results in comparison to the test results of the group R1.7 conducted at different levels of axial force (from 0.05 to 0.5). It can be seen that the shear strength increased gradually by around 11.1%, 9.6% and 8.56% (in the mean) when the axial force was increased from 0 to 0.1, 0.1 to 0.2 and 0.2 to $0.35f'_c A_g$, respectively. However, there were only increase of around 1.3% and 1.5% when the axial force increases to 0.4 and $0.5f'_c A_g$. Experiments of this group (Tran (2010)⁽¹⁹⁾) also observed similar results where shear strength was almost unchanged when the applied axial force was beyond the level of $0.35f'_c A_g$.

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Figure 6.17 Influence of axial force to shear strength

From the above discussion, considering that ASCE 41 provides the closest estimations for shear strength when axial force is applied up to $0.35f_c A_g$, modified concrete contribution to shear strength at a high level of axial force can be written as (all units are in MPa):

$$V_c = \frac{0.5\sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{P}{0.5\sqrt{f'_c} A_g}} 0.8A_g = \frac{0.5\sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{P}{f'_c A_g} * \frac{\sqrt{f'_c}}{0.5}} 0.8A_g \quad (6.5)$$

The upper limit of V_c is:

$$[V_c]_{max} = \frac{0.5\sqrt{f'_c}}{M/Vd} \sqrt{1 + 0.7\sqrt{f'_c}} 0.8A_g \quad (6.6)$$

and the upper limit of shear strength is:

$$[V_n]_{max} = [V_c]_{max} + V_s = \frac{0.5\sqrt{f'_c}}{M/Vd} \sqrt{1 + 0.7\sqrt{f'_c}} 0.8A_g + \frac{A_v f_y d}{s} \quad (6.7)$$

The proposed modified ASCE 41 equation for shear strength is therefore written as:

$$V_n = \begin{cases} k_1 \frac{A_v f_y d}{s} + \lambda k_2 \left(\frac{0.5\sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{P}{0.5\sqrt{f'_c} A_g}} \right) 0.8A_g & \text{for } P < 0.35f'_c A_g \\ k_1 \frac{A_v f_y d}{s} + \lambda k_2 \frac{0.5\sqrt{f'_c}}{M/Vd} \sqrt{1 + 0.7\sqrt{f'_c}} 0.8A_g & \text{for } P \geq 0.35f'_c A_g \end{cases} \quad (6.8)$$

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 and λ is equal to 1 for normal-weight concrete.

Applying the proposed equation (6.8) to previous tests on columns with high axial loads ($P \geq 0.35f'_cA_g$), the results are shown in **Figure 6.17** and **Table 6.4**. It was found that the average ratio of the predicted to the test results for shear strength by the proposed equation was 1.02 while ASCE 41 gave 1.19. The coefficients of variation were 0.33 and 0.43 for the proposed method (6.8) and ASCE 41 method, respectively. The comparisons clearly show that the proposed equation produces a better estimation of shear strength for RC columns when high axial force is applied.

Figure 6.18 Comparisons of shear strength between proposed equation (6.8) and ASCE 41

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Table 6.4 Comparisons of shear strength between proposed equation and ASCE 41

Authors	Specimen	b (mm)	h (mm)	f_c (Mpa)	f_{yt} (Mpa)	A_v (mm ²)	s (mm)	a/d	$\frac{P}{f_c A_g}$	V_n _{ASCE}	V_n _{Pro}	V_{test}	$\frac{V_{pro}}{V_{test}}$	$\frac{V_{ASCE}}{V_{test}}$
Tran 2010 ⁽¹⁹⁾	RC-1.7- 0.35	250	490	26.8	393	56.5	125	1.9	0.5	408.2	363.7	355.2	1.02	1.15
Sezen and Moehle 2006 ⁽¹⁷⁾	Specimen 2	457	457	21.1	476	308.0	305	3.8	0.6	452.4	399.8	356.2	1.12	1.27
	X-1	267	267	35.7	273	37.7	164	1.6	0.60	323.9	261.2	302.4	0.86	1.07
	X-2	267	267	35.7	273	37.7	164	1.6	0.40	302.4	287.3	327.6	0.88	0.92
Lam <i>et al.</i> 2003 ⁽⁷³⁾	X-3	267	267	35.7	273	37.7	55	1.6	0.40	337.0	321.9	305.6	1.05	1.10
	X-8	267	267	35.7	270	37.7	100	1.6	0.40	313.3	298.2	259.6	1.15	1.21
	X-9	267	267	35.7	270	37.7	100	1.6	0.40	313.3	298.2	305.1	0.98	1.03
Amitsuet <i>al.</i> 1991 ⁽⁵⁵⁾	CB060C	278	278	46.3	441	56.5	52	1.4	0.74	627.2	485.1	505.0	0.96	1.24
Arakawa <i>et al.</i> 1989 ⁽⁵⁶⁾	OA5	180	180	33.0	249	56.5	64	1.4	0.45	163.5	151.4	133.6	1.13	1.22
Ono <i>et al.</i> 1989 ⁽⁷⁴⁾	CA060C	200	200	25.8	426	56.5	70	1.9	0.62	171.3	147.1	134.6	1.09	1.27
	No. 806	80	80	32.3	341	25.1	80	1.3	0.6	38.8	32.3	31.6	1.02	1.23
Zhou <i>et al.</i> 1985 ⁽⁷⁵⁾	No. 1007	80	80	34.0	341	25.1	80	1.3	0.7	42.3	33.2	36.7	0.91	1.15
	No. 1309	80	80	32.8	341	25.1	80	1.3	0.9	45.6	32.6	29.6	1.10	1.54
													Mean	1.02 1.19
													COV	0.33 0.43

6.4 Summary

An experimental program was carried out on seven reinforced concrete columns with deformed rebars and light transverse reinforcement under simulated gravity and different directions of seismic loading. Numerical studies were conducted to supplement test results and to further investigate the effects of seismic loading directions and axial force. The conclusions drawn are as follows:

1. The loading direction had significant effects on axial failure modes and the angle of critical diagonal cracks. It was of key importance to the drift ratio at axial failure and the maximum energy dissipation capacity of test specimens. However, it did not have any significant effect on the initial stiffness and the drift ratio at maximum shear force.
2. The shear strength of columns in non-principal directions can be estimated once their shear strength in the principal direction is found; the results matched the circle-interaction line for square columns and the ellipse-interaction line for rectangular ones.
3. There were two modes of axial failure observed in the test specimens. The development of critical shear cracks caused failure in the first mode, while the second failure mode resulted from vertical bond splitting crack. Existing models to estimate the drift ratio at axial failure were found to be not applicable to those columns that had failed in the second mode.
4. An upper limit to shear strength was proposed by modifying the contribution of the concrete part in ASCE 41 when the applied axial force was higher than $0.35f'_cA_g$.

CHAPTER VII: SUMMARY, CONCLUSIONS AND OUTLOOK FOR FUTURE STUDIES

7.1 Summary and Conclusions

Seismic behavior of existing RC columns has attracted great attention from the research community involved in modern earthquake engineering. This study contributes further to understanding the seismic response of existing RC columns, particularly with light transverse reinforcement and plain or deformed longitudinal reinforcement. The study also assists the development of a shear strength model that reasonably expresses the failure mechanism and adequately estimates the shear strength of RC columns.

An experimental program consisting of two test series was conducted to assess the seismic performance of existing RC columns. **Series 1** was carried out on nine one-half scale column specimens having plain longitudinal reinforcing bars and limited stirrups. The specimens were tested under simulated gravity and seismic loadings. The variables of interest were the levels of axial load, the shear span to depth ratio and the cross-sectional shape. Three levels of axial load, $0.2f'_cA_g$, $0.35f'_cA_g$ and $0.5f'_cA_g$, were investigated. The specimens were designed to have shear span to depth ratios of 1.4, 1.7 or 2.4 and to have cross-sectional shape of wall-like columns, rectangular columns or square columns. Experimental results showed that all columns with plain rebars failed with the development of vertical cracks forming along the longitudinal reinforcing bars located at the mid-height of the column cross sections. These vertical cracks developed at smaller drift ratios in columns under higher levels of axial load. This type of cracks did not substantially affect column's initial stiffness.

Unlike columns with plain rebars tested as cantilevers, the columns tested under double curvature bending did not experience remarkable slip deformation. Instead, the deformation resulted from sliding movements of the column body along the vertical splitting cracks and is believed to contribute significantly to the energy

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dissipation capacity. Effects of shear span to depth ratios, axial forces and cross-sectional shapes on the performance of columns with plain rebars were similar to how those parameters affect columns with deformed rebars. Most remarkably, shear strength of columns with plain rebars benefits from an increase in axial forces up to $0.35f'_cA_g$. In other words, when the axial force was increased above $0.35f'_cA_g$ the shear strength was almost unchanged.

A comparison was made between the test results of the columns with plain rebars in Series 1 and the test results of similar columns reinforced with deformed rebars in previous study. It was found that, unlike columns with deformed rebars, most columns with plain rebars did not develop flexural yielding and also did not experience the yielding of transverse reinforcement before the maximum shear force was reached. Columns with deformed rebars produce higher shear strength than columns with plain rebars, ranging from 22% to 33% depending on cross sectional shapes but regardless of the levels of applied axial force or shear span to depth ratios. However, columns with plain rebars generally possess higher initial stiffness and significantly higher energy dissipation capacity. Under moderate levels of applied axial force, the failure mechanism might change from diagonal shear failure to splitting failure when deformed rebars were replaced by plain longitudinal reinforcement.

A further comparison was made between the test results of the Series 1 with the analytical predictions by the current assessment code ASCE 41. The comparison showed that ASCE 41 inconsistently and substantially over-estimates the initial stiffness of RC columns with plain rebars. Drift capacities are not predicted well at any acceptable level. More importantly, the guideline gives an over-estimation of the shear strength for columns with plain rebars that is proved to be unsafe in practice. From this research, it is highly recommended, that a modification factor of 0.70 should multiply to the shear strengths estimated from Equation (6-4) in ASCE 41 when evaluating columns with plain rebars. This finding was verified with previous tests on RC columns with plain rebars and showed good matching.

Observations from the tests of Series 1 and previous tests showed that a large number of test columns failed by splitting failure. This resulted from the development of vertical cracks along the mid-height at the central region of the specimen. This type of failure was found only in columns with a shear span to depth ratio not greater than 2. This phenomenon appeared in both columns with deformed rebars and plain rebars, and was not directly related to the level of applied axial compression force. Columns that failed by splitting failure reached a lower shear capacity but a higher drift capacity compared to those failed by diagonal shear failure.

In this study, a new load-transfer path was proposed to explain that splitting failure was due to the vertical sliding shear stress exceeding the ultimate splitting strength along vertical potential cracks. A new shear strength model was formulated based on the proposed failure mechanism. The proposed analytical model was validated using the data of 17 column tests in which splitting cracks were observed. Good agreement was obtained with a mean test-to-predicted ratio of 1.05 and a coefficient of variation of 0.135. The model also yielded good correlations with observed behavior. The extended model could be used to estimate the shear strength of short columns (having shear span to depth ratio not higher than 2.5). The formula is expressed as follows:

$$V_n = 0.9 \frac{L-h}{s} A_v f_{yt} e^{-0.9\rho_v} - 0.166 \left[n^{0.1} + 0.2 \left(\frac{a}{d} \right)^{-2} \right] (L-h)b \sqrt{f'_c} + (0.08 + 0.064n) f'_c A_g \quad (7.1)$$

where V_n (N) is the estimated shear strength, L (mm) is column height; h (mm) and b (mm) are the depth and width of column section, respectively; $A_g = b \times h$ (mm²); a (mm) is the shear span, d (mm) is the distance from the extreme compression fiber to the centroid of tensile reinforcement; A_v (mm²), f_{yt} (MPa), ρ_v (%) and s (mm) are total area (in one direction, within spacing s) the ratio, yield strength and spacing of transverse reinforcement, respectively; f'_c (MPa) is the compressive strength of concrete; n is the level of axial force which is calculated as $P/f'_c A_g$ with P (N) is the applied axial force.

Verification of the proposed shear strength model (Equation 7.1) with experimental data collected from 89 RC short columns showed a better agreement than the models of ACI 318 (2011), ASCE 41 (2007) and Biskinis et al. (2004). The proposed model suggests that, for short columns, the transverse reinforcement index ($\rho_v f_{yt}$) could be used effectively (for improving shear strength) and economically up to 3.1MPa.

Another test series, namely **Series 2**, consisting of seven column specimens having light transverse reinforcement and deformed reinforcing bars was conducted. This investigation aimed to study the effects of loading directions on seismic behavior of existing RC columns. Two square columns with shear span to depth ratio of 1.7 and two square columns with that ratio of 2.4 were tested under loading directions of either 30° or 45° with an applied axial force of $0.2f'_c A_g$. Three rectangular columns were tested under loading directions of 30°, 45° and 60° (from its strongest direction) with an applied axial force of $0.35f'_c A_g$. The performance of each specimen was examined in terms of cracking patterns, hysteretic response, initial stiffness, shear strength, drift capacity and energy dissipation capacity. The results obtained from the tests on diagonal directions in this study were compared with previous results on similar columns tested under principal loading direction. The comparisons highlighted that the loading direction had significant effects on axial failure modes and angle of critical diagonal cracks. It was an important factor influencing to the drift ratio at axial failure and energy dissipation capacity of the test specimens. However, it did not have any significant effect on the initial stiffness and the drift ratio at maximum shear force. The shear strength of columns in non-principal directions can be estimated once their shear strength in the principal direction was found; the results matched the circle-interaction line for square columns and the ellipse-interaction line for rectangular ones. There were two modes of axial failure observed from the test specimens. The development of critical diagonal shear cracks caused the failure in the first mode, while vertical splitting cracks resulted in the failure in the second failure mode.

A finite element analysis was employed by using fiber elements with the Sakai and Kawashima's concrete model in Engineer's Studio program, which was found as a

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suitable numerical model for predicting the seismic behavior of existing RC columns. The numerical simulations produced good agreement with the observed behavior, including peak strength, post-peak response, and drift ratios at axial failure. The noticeable discrepancies were related to the displacement corresponding to the peak lateral load, drift ratio at axial force and shear resistance capacity at final failure. Similarities between the test results of Series 2 and the numerical study suggested an upper limit of shear strength contributed by concrete which was proposed by modifying the contribution of the concrete part in ASCE 41 when the applied axial force was higher than $0.35f'_cA_g$. The proposed equation is shown as follows:

$$V_n = \begin{cases} k_1 \frac{A_v f_y d}{s} + \lambda k_2 \left(\frac{0.5\sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{P}{0.5\sqrt{f'_c} A_g}} \right) 0.8A_g & \text{for } P < 0.35f'_c A_g \\ k_1 \frac{A_v f_y d}{s} + \lambda k_2 \frac{0.5\sqrt{f'_c}}{M/Vd} \sqrt{1 + 0.7\sqrt{f'_c}} 0.8A_g & \text{for } P \geq 0.35f'_c A_g \end{cases} \quad (7.2)$$

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 and λ is equal to 1 for normal-weight concrete; M and V are the moment and shear at the section of maximum moment, respectively. Other parameters are defined the same as in Equation (7.1).

7.2 Recommendations for Future Studies

Several ideas for continuing the study presented in this dissertation have arisen during the development of the research. They are summarized in the following paragraphs.

- Experiments have shown that columns with plain rebars have significantly lower shear strength compared to columns with deformed rebars. The differences depend on the depth to width ratio of the column cross section. However, due to the

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limited number of the tests conducted, there is not enough data to develop a more general formula for predicting the shear strength of RC columns with plain rebars. Therefore, more experimental studies on such columns with similar variables as in this study are suggested to investigate the influences of column's parameters to the shear strength of columns with plain rebars. It would also be useful to conduct parametric studies using a numerical approach taking into account the effect of plain surface of longitudinal reinforcement.

- Further experimental and analytical studies on the splitting failure mechanism are recommended. In the context of an experimental study, the same tests could be used to increase the number of specimens in database to validate the proposed shear strength model (presented in Chapter 5) with the aim of predicting shear strength for RC columns having shear span to depth ratio not greater than 2.5. A model capable of predicting diagonal shear failure or vertical splitting failure is strongly suggested for further research.

- This study shows that the shear strength of RC columns only benefits from axial force when the level of compressive force, $P/f'_c A_g$, does not exceed 0.35. Parametric studies in Chapter V shows that shear strength of short columns only benefits from transverse reinforcement when its transverse reinforcement index ($\rho_v f_{yt}$) does not exceed 3.1 MPa. Further experimental and analytical studies to verify these two finding for all RC columns would be very useful to improve current design and assessment codes.

- The effect of loading directions on the seismic response of RC columns with deformed rebars has been investigated in this study. However the seismic behavior of RC columns with plain rebars under different loading directions has not been found in previous studies. While understanding that behavior is useful for seismic assessment of existing building, a similar methodology as the experimental study in Chapter 6 could be used to investigate such an effect on RC columns with plain rebars.

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5. **Thanh-Phuong Pham** and Bing Li “Seismic Performance Assessment of L-Shaped Reinforced Concrete Columns” *ACI Structural Journal*, (Under Peer Review).

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6. Bing Li and **Thanh-Phuong Pham** “Seismic Behavior of RC Columns with Light Transverse Reinforcement Under Different Loading Direction” Proceedings of the Ninth Pacific Conference on Earthquake Engineering Building an Earthquake-Resilient Society, 14-16 April, 2011, Auckland, New Zealand.
7. Bing Li and **Thanh-Phuong Pham** “Influence of Seismic Loading Direction on the Response of Non-Ductile RC Columns” Proceedings of The 13th Taiwan-Korea-Japan Joint Seminar on Earthquake Engineering for Building Structures SEEBUS 2011, South Korea, November 11-12, 2011.
8. Bing Li and **Thanh-Phuong Pham** “Response of Old-type RC Columns under Seismic Loading” Proceedings of The 14th Taiwan-Korea-Japan Joint Seminar on Earthquake Engineering for Building Structures SEEBUS 2012, Osaka, Japan, November 1-3, 2012.
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APPENDIX

A1. Details of Test Specimens and Instrumentations

Test Specimens Series 1

Specimens SP1-1.7-0.2, SP2-1.7-0.35.....Drawing S1.01

Specimens SP2-2.4-0.2, SP3-2.4-0.35.....Drawing S1.02

Specimens SR1-1.7-0.2, SR2-1.7-0.35.....Drawing S1.03

Specimens SW1-1.4-0.2, SW1-1.4-0.35, SW1-1.4-0.5.....Drawing S1.04

Test Specimens Series 2

Specimens SC2.4-30.....Drawing S2.01

Specimens SC2.4-45.....Drawing S2.02

Specimens SC1.7-30.....Drawing S2.03

Specimens SC1.7-45.....Drawing S2.04

Specimens SR1.7-30.....Drawing S2.05

Specimens SR1.7-45.....Drawing S2.06

Specimens SR1.7-60.....Drawing S2.07

Strain Gauge Arrangements

Strain Gauge Arrangement for Series 1.....Drawing SG.01

Strain Gauge Arrangement for Series 2.....Drawing SG.02

A2. Test Results of Specimens in Series 1

Specimen SP1-1.7-0.2

Figure A.1 Hysteretic response of specimen SP1-1.7-0.2

Figure A.2 Observed cracking patterns of specimen SP1-1.7-0.2

Appendix

Figure A.3 Strain in longitudinal Reinforcing bars of specimen SP1-1.7-0.2

Figure A.4 Strain in transverse reinforcement of specimen SP1-1.7-0.2

Appendix

Figure A.5 Displacement decomposition of Specimen SP1-1.7-0.2

Figure A.6 Cumulative energy dissipation of specimen SP1-1.7-0.2

Specimen SP2-1.7-0.35

Figure A.7 Hysteretic response of specimen SP1-1.7-0.35

Figure A.8 Observed cracking patterns of specimen SP2-1.7-0.35

Appendix

Figure A.9 Strain in longitudinal reinforcing bars of specimen SP2-1.7-0.35

Figure A.10 Strain in transverse reinforcement of specimen SP2-1.7-0.35

Figure A.11 Displacement decomposition of Specimen SP2-1.7-0.35

Figure A.12 Cumulative energy dissipation of specimen SP2-1.7-0.35

Specimen SP3-2.4-0.2

Figure A.13 Hysteretic response of specimen SP3-2.4-0.2

Figure A.14 Observed cracking patterns of specimen SP3-2.4-0.2

Appendix

Figure A.15 Strain in longitudinal reinforcing bars of specimen SP3-2.4-0.2

Figure A.16 Strain in transverse reinforcement of specimen SP3-2.4-0.2

Figure A.17 Displacement decomposition of Specimen SP3-2.4-0.2

Figure A.18 Cumulative energy dissipation of specimen SP3-2.4-0.2

Specimen SP4-2.4-0.35

Figure A.19 Hysteretic response of specimen SP4-2.4-0.35

Figure A.20 Observed cracking patterns of specimen SP4-2.4-0.35

Appendix

Figure A.21 Strain in longitudinal reinforcing bars of specimen SP4-2.4-0.35

Figure A.22 Strain in transverse reinforcement of specimen SP4-2.4-0.35

Appendix

Figure A.23 Displacement decomposition of Specimen SP4-2.4-0.35

Figure A.24 Cumulative energy dissipation of specimen SP4-2.4-0.35

Specimen SR1-1.7-0.35

Figure A.25 Hysteretic response of specimen SR1-1.7-0.35

Figure A.26 Observed cracking patterns of specimen SR1-1.7-0.35

Appendix

Figure A.27 Strain in longitudinal reinforcing bars of specimen SR1-1.7-0.35

Figure A.28 Strain in transverse reinforcement of specimen SR1-1.7-0.35

Figure A.29 Displacement decomposition of Specimen SR1-1.7-0.35

Figure A.30 Cumulative energy dissipation of specimen SR1-1.7-0.35

Specimen SR2-1.7-0.5

Figure A.31 Hysteretic response of specimen SR2-1.7-0.5

Figure A.32 Observed cracking patterns of specimen SR2-1.7-0.5

Appendix

Figure A.33 Strain in longitudinal reinforcing bars of specimen SR2-1.7-0.5

Strains in transverse reinforcement

Figure A.34 Strain in transverse reinforcement of specimen SR2-1.7-0.5

Figure A.35 Displacement decomposition of Specimen SR2-1.7-0.5

Figure A.36 Cumulative energy dissipation of specimen SR2-1.7-0.5

Specimen SW1-1.4-0.2

Figure A.37 Hysteretic response of specimen SW1-1.4-0.2

Figure A.38 Observed cracking patterns of specimen SW1-1.4-0.2

Appendix

Figure A.39 Strain in longitudinal reinforcing bars of specimen SW1-1.4-0.2

Strains in transverse reinforcement

Figure A.40 Strain in transverse reinforcement of specimen SW1-1.4-0.2

Appendix

Figure A.41 Displacement decomposition of Specimen SW1-1.4-0.2

Figure A.42 Cumulative energy dissipation of specimen SW1-1.4-0.2

Specimen SW2-1.4-0.35

Figure A.43 Hysteretic response of specimen SW2-1.4-0.35

Figure A.44 Observed cracking patterns of specimen SW2-1.4-0.35

Appendix

Figure A.45 Strain in longitudinal reinforcing bars of specimen SW2-1.4-0.35

Figure A.46 Strain in transverse reinforcement of specimen SW2-1.4-0.35

Figure A.47 Displacement decomposition of Specimen SW2-1.4-0.35

Figure A.48 Cumulative energy dissipation of specimen SW2-1.4-0.35

Specimen SW3-1.4-0.5

Figure A.49 Hysteretic response of specimen SW3-1.4-0.5

Figure A.50 Observed cracking patterns of specimen SW3-1.4-0.5

Appendix

Figure A.51 Strain in longitudinal reinforcing bars of specimen SW3-1.4-0.5

Figure A.52 Strain in transverse reinforcement of specimen SW3-1.4-0.5

Appendix

Figure A.53 Displacement decomposition of Specimen SW3-1.4-0.5

Figure A.54 Cumulative energy dissipation of specimen SW3-1.4-0.5

A3. Test Results of Specimens in Series 2

Specimen S2.4-30

Figure A.55 Hysteretic response of specimen S2.4-30

Figure A.56 Observed cracking patterns of specimen S2.4-30

Appendix

Figure A.57 Strain in longitudinal reinforcing bars of specimen S2.4-30

Figure A.58 Strain in transverse reinforcement of specimen S2.4-30

Figure A.59 Cumulative energy dissipation of specimen S2.4-30

Specimen S2.4-45

Figure A.60 Hysteretic response of specimen S2.4-45

Figure A.61 Observed cracking patterns of specimen S2.4-45

Appendix

Figure A.62 Strain in longitudinal reinforcing bars of specimen S2.4-45

Figure A.63 Strain in transverse reinforcement of specimen S2.4-45

Appendix

Figure A.64 Cumulative energy dissipation of specimen S2.4-45

Specimen S1.7-30

Figure A.65 Hysteretic response of specimen S1.7-30

Figure A.66 Observed cracking patterns of specimen S1.7-30

Appendix

Figure A.67 Strain in longitudinal reinforcing bars of specimen S1.7-30

Figure A.68 Strain in transverse reinforcement of specimen S1.7-30

Figure A.69 Cumulative energy dissipation of specimen S1.7-30

Specimen S1.7-45

Figure A.70 Hysteretic response of specimen S1.7-45

Figure A.71 Observed cracking patterns of specimen S1.7-45

Appendix

Figure A.72 Strain in longitudinal reinforcing bars of specimen S1.7-45

Figure A.73 Strain in transverse reinforcement of specimen S1.7-45

Figure A.74 Cumulative energy dissipation of specimen S1.7-45

Specimen R1.7-30

Figure A.75 Hysteretic response of specimen R1.7-30

Figure A.76 Observed cracking patterns of specimen R1.7-30

Appendix

Figure A.77 Strain in longitudinal reinforcing bars of specimen R1.7-30

Figure A.78 Strain in transverse reinforcement of specimen R1.7-30

Figure A.79 Cumulative energy dissipation of specimen R1.7-30

Specimen R1.7-45

Figure A.80 Hysteretic response of specimen R1.7-45

Figure A.81 Observed cracking patterns of specimen R1.7-45

Appendix

Figure A.82 Strain in longitudinal reinforcing bars of specimen R1.7-45

Figure A.83 Strain in transverse reinforcement of specimen R1.7-45

Figure A.84 Cumulative energy dissipation of specimen R1.7-45

Specimen R1.7-60

Figure A.85 Hysteretic response of specimen R1.7-60

Figure A.86 Observed cracking patterns of specimen R1.7-60

Appendix

Figure A.87 Strain in longitudinal reinforcing bars of specimen R1.7-60

Figure A.88 Strain in transverse reinforcement of specimen R1.7-60

Appendix

Figure A.89 Cumulative energy dissipation of specimen R1.7-60

A4. FEA Results for Specimens in Series 2

Appendix

Appendix

Appendix

Figure A.90 Comparison of hysteretic responses between Test and FEM for Series 2