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**NANYANG
TECHNOLOGICAL
UNIVERSITY**

**SHEAR BEHAVIOUR OF HIGH STRENGTH
CONCRETE WALLS SUBJECTED TO CYCLIC
LATERAL LOADING**

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
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**SHEAR BEHAVIOUR OF HIGH STRENGTH
CONCRETE WALLS SUBJECTED TO CYCLIC
LATERAL LOADING**

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SUMMARY

This study focuses on investigating the shear behaviour of high strength concrete (HSC) walls with concrete compressive strength about 100 MPa. Furthermore, the other purpose of this study is to develop analytical models based on truss analogy for predicting the shear strength of RC walls.

Literature review and data collection of past experiments on RC walls have been done. From the data collected, it can be concluded that there are very few data of experiments on RC walls failing in shear with concrete compressive strength about 100 MPa or higher. Moreover, since the shear behaviour is also much more complicated as compared to flexure behaviour, it is therefore necessary to conduct an experimental study on such HSC walls to investigate the shear behaviour since experimental study is the best representation of real structures as compared to numerical study using computer models.

There are seven reduced-scale specimens designed for such purpose. Some significant parameters that affect the shear strength of RC walls are investigated such as wall height to length ratio, vertical and horizontal web reinforcement ratios, and presence of boundary elements. The specimens are subjected to static vertical axial loading and cyclic lateral in-plane loading which are typical loading conditions for RC walls. The experimental results show that the shear strength of RC walls is greatly affected by wall height to length ratio. Furthermore, both vertical and horizontal web reinforcement have contribution to the shear strength of RC walls and their effectiveness depends on wall height to length ratio. In addition, presence of boundary elements can significantly increase the shear strength of RC walls.

For the analytical work, in this study, analytical models have been developed based on the truss analogy to predict the shear strengths of RC walls. The proposed models are intended to serve as accurate methods that can be used directly by engineers to predict the shear strengths of RC walls. Results from this experimental study are used as the basis for certain assumptions in the models. Data available from past experiments on RC walls failing in shear as well as from this experimental study are used to verify the accuracy of the proposed model

predictions. Furthermore, building codes and other analytical model predictions are included as well for comparison. The analysis results show that the predictions of the proposed models are on average closer to experimental shear strength of RC walls while still being on the conservative side. They have lower coefficient of variation as compared to other predictions. In addition, the proposed models can predict the shear strength of RC walls with consistent accuracy for wide ranges of wall height to length ratios and concrete compressive strengths which is not the case for building codes and other analytical model predictions.

LIST OF PUBLICATIONS

Chandra, J., Y. Liu and Teng, S. (2011). Analytical Study on High Strength Concrete Shear Walls. 36th Conference on Our World in Concrete and Structures. Singapore: 221-230.

Chandra, J. and Teng, S. (2012). Shear Strength of Normal to High Strength Concrete Walls. The 1st International Conference on Sustainable Civil Engineering Structures and Construction Materials. Yogyakarta, Indonesia, Department of Civil and Environmental Engineering, Gadjah Mada University: 138-145.

Teng, S. and Chandra, J. (2016). Cyclic Shear Behavior of High Strength Concrete Structural Walls. ACI Structural Journal, 113(6), 1335-1345.

Chandra, J. and Teng, S. Truss Model for Shear Strength of Structural Concrete Walls. ACI Structural Journal (under review).

The paper that was presented at 36th Conference on Our World in Concrete and Structures, Singapore has been awarded JCI-OWICS Award 2011 from the Japan Concrete Institute.

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LIST OF SYMBOLS

a_s	Depth of diagonal concrete strut
A_{cv}	Gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered
A_g	Gross cross sectional area
A_s	Area of reinforcement
A_{sb}	Area of vertical reinforcement in boundary element
A_{sl}	Area of longitudinal web reinforcement
A_{st}	Area of transverse web reinforcement
A_{str}	Area of diagonal concrete strut
A_w	Area of web
b	Width of web cross section
b_{ef}	Effective width of boundary element
b_f	Width of boundary element
b_s	Width of diagonal concrete strut
c	Depth of compression zone at the bottom of wall
c_{1-4}	Constants to be determined
C	Compression force in the compression zone
$C_{Rd,c}$	Function of resistance design properties of concrete
d	Effective depth of cross section
d_{be}	Equivalent bar diameter
d_w	Effective depth of wall
D	Compression force in diagonal concrete strut
D_n	Nominal strength of diagonal concrete strut
D_u	Dowel force of vertical reinforcement in one boundary element
E_c	Modulus elasticity of concrete
E_s	Modulus elasticity of steel reinforcement
f'_c	Concrete cylinder compressive strength
f'_{ct}	Concrete cylinder tensile strength
f_{cr}	Cracking stress of concrete
f_{ct}	Concrete cylinder splitting tensile strength

f_h	Stress in horizontal web reinforcement
f_l	Stress in longitudinal web reinforcement
f_p	Maximum softened compressive stress of concrete
f_t	Stress in transverse web reinforcement
f_v	Stress in vertical web reinforcement
f_y	Yield strength of reinforcement
f_{yb}	Yield strength of vertical reinforcement in boundary element
f_{yh}	Yield strength of horizontal web reinforcement
f_{yl}	Yield strength of longitudinal web reinforcement
f_{yt}	Yield strength of transverse web reinforcement
f_{yv}	Yield strength of vertical web reinforcement
F_h	Tension force in horizontal tie
F_v	Tension force in vertical tie
H_w	Wall height
H'	Distance measured from point of application of external shear force to wall base
k	Coefficient or factor for calculating the shear resistance of concrete
k_1	Coefficient or factor for accounting axial force contribution to the shear resistance of concrete
K	Strut and tie index or reinforcement index
K_c	Coefficient reflecting influence of transverse tensile straining
K_f	Coefficient reflecting influence of nominal strength of concrete
ℓ	Internal level arm of the couple forces at wall base
L_w	Wall length
M	Applied bending moment at wall
N	Applied axial force at boundary element
P	Applied axial force at wall
r	Distance measured from point of application of resultant force to nearest wall edge
R	Resultant force of external axial force and tension force in tension tie
R_d	Wall shear ratio resisted by diagonal mechanism
R_h	Wall shear ratio resisted by horizontal mechanism

R_v	Wall shear ratio resisted by vertical mechanism
s	Spacing of reinforcement
t_f	Thickness of boundary element
t_w	Thickness of wall web
T	Tension force in tension tie
T_n	Nominal strength of steel tension tie
V	Applied shear force at wall
V_c	Concrete contribution to overall shear strength
V_{exp}	Maximum lateral in-plane load applied
V_n	Nominal strength
V_{nf}	Nominal flexure strength
V_{ns}	Nominal shear strength
$V_{n,max}$	Nominal maximum shear strength value
V_s	Steel reinforcement contribution to overall shear strength
V_{wh}	Horizontal wall shear force
V_{wv}	Vertical wall shear force
x_1	Wall parameter
x_2	Wall parameter
z	Inner lever arm, for a member with constant depth, corresponding to the bending moment in the element under consideration
α	Angle of diagonal concrete strut to longitudinal axis
α_c	Coefficient defining the relative contribution of concrete strength to nominal wall shear strength
α_{cw}	Coefficient taking account of the state of the stress in the compression chord
α_w	Coefficient taking account of confinement effect of web reinforcement to concrete strut strength, related to spacing of web reinforcement
β	Coefficient for calculating the shear strength of concrete
γ	Shear deformation
γ_c	Partial factor for concrete
γ_{lt}	Shear strain in the element in l-t coordinate system
γ_{vh}	Shear strain in the element in v-h coordinate system
Δ	Deformation of wall

Δ_F	Flexure deformation of wall
Δ_S	Shear deformation of wall
Δ_{SL}	Sliding shear deformation of wall
ε_{cr}	Cracking strain of concrete
ε_d	Principal strain in d axis
ε_h	Normal strain in horizontal axis
ε_l	Strain in longitudinal axis
ε_o	Strain of concrete at maximum compressive stress
ε_p	Strain of softened concrete at maximum softened compressive stress
ε_r	Principal strain in r axis
ε_s	Strain in steel
ε_t	Strain in transverse axis
ε_v	Normal strain in vertical axis
ε_y	Yield strain of reinforcement
ε_{yh}	Yield strain of horizontal web reinforcement
ε_{yv}	Yield strain of vertical web reinforcement
ζ	Softening factor of concrete
θ	Angle of inclination of diagonal concrete strut with respect to horizontal axis
θ_s	Angle between concrete compression strut and member axis perpendicular to shear force
θ_{sa}	Angle of compression strut in arch mechanism
θ_{st}	Angle of compression strut in truss mechanism
λ	Softening coefficient of concrete
λ'	Modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normal weight concrete of the same compressive strength
v_1	Strength reduction factor for concrete cracked in shear
v_e	Effective factor of concrete compressive strength
ρ	Reinforcement ratio
ρ_b	Ratio of vertical reinforcement in boundary element
ρ_h	Horizontal web reinforcement ratio

ρ_l	Longitudinal web reinforcement ratio
ρ_t	Transverse web reinforcement ratio
ρ_v	Vertical web reinforcement ratio
σ_{cp}	Compressive stress in concrete from axial load or prestressing
σ_d	Principal stress in d axis
$\sigma_{d,max}$	Maximum compressive stress acting on nodal zone
σ_h	Normal stress in horizontal axis
σ_l	Normal stress in longitudinal axis
σ_r	Principal stress in r axis
σ_t	Normal stress in transverse axis
σ_v	Normal stress in vertical axis
τ	Average shear stress
τ_{lt}	Average shear stress in l-t coordinate system
τ_{rd}	Average shear stress in r-d coordinate system
τ_{vh}	Average shear stress in v-h coordinate system
ϕ	Angle which tangent is the shear span ratio of wall
ω_w	Coefficient taking account of confinement effect of web reinforcement to concrete strut strength, related to ratio of web reinforcement

CHAPTER 1

INTRODUCTION

1.1 General Background

Concrete is one of the most popular building materials used nowadays. General availability of the basic ingredients of concrete, such as sand, gravel, and cement as well as the cost of these materials which is relatively low as compared to other building materials make concrete the often preferred choice of main material to be used for structural components. Furthermore, its combination with steel bars to form reinforced concrete (RC) and its flexibility to be cast into almost any shape are very useful for construction practices.

As world population is growing, people start to build more buildings than before and thus increase the demand for new land to build the buildings. This demand may cause a problem in a country with small area like Singapore. Hence, space creation is needed to accommodate the demand and solve the problem. In past years, many methods to create more space have been developed by researchers and engineers. One recent method is to build underwater structures.

This recent method is basically almost the same as land reclamation. It aims to expand new land laterally towards offshore. The difference is that instead of filling the sea space with soil in the case of land reclamation, this method purposely builds underwater structures to cover the newly created land and prevent seawater to penetrate. Therefore, it can create more space than that of land reclamation since the space below seawater level can also be utilized for some purposes.

For construction of large underwater structures with desired service period more than hundred years, concrete seems to be the most suitable material. In this case, high strength concrete (HSC) with high durability and low permeability is needed to prevent corrosion of steel bars and ensure the durability of the structures. Moreover, HSC with high stiffness can lead to reduction of the size of structural members, such as columns and walls, which benefits in creating more space for other purposes.

Concrete structural wall is one of the key structural elements in underwater structures as well as many other structures. The use of structural wall has been widely recognized long time ago, especially for resisting lateral loadings, such as wind and earthquake (Fintel 1991). In addition, many structural walls are optimized to resist gravity loadings as well. In underwater structures, the walls can be utilized to resist gravity loadings from the upper structures as well as lateral loadings from earthquake, sea waves, tsunami, accidental impact load from ships, etc.

Research about structural wall began as early as in 1950s (Galletly 1952; Benjamin and Williams 1957; Hirose 1957). Since then many experimental studies (Cardenas and Magura 1972; Yamada et al. 1974; Wang et al. 1975; Barda et al. 1977; Vallenat et al. 1979; Cardenas et al. 1980; Corley et al. 1981; Moehle 1984; Tomii and Esaki 1984; Lefas et al. 1990; Wallace and Moehle 1992; Pilakoutas and Elnashai 1995; Mo and Chan 1996; Gupta and Rangan 1998; Kabeyasawa and Hiraishi 1998; Mo and Kuo 1998; Salonikios et al. 1999; Zhang and Wang 2000; Sittipunt et al. 2001; Hidalgo et al. 2002; Chiou et al. 2003; Mosalam et al. 2003; Liao et al. 2004; Massone and Wallace 2004; Yun et al. 2004; Li 2005; Deng et al. 2008; Farvashany et al. 2008; Kuang and Ho 2008; Yan et al. 2008; Liang et al. 2013; Burgueno et al. 2014; Li et al. 2015; Tran and Wallace 2015) have been conducted to investigate further the behaviour of structural wall which is also well known as shear wall.

In order to construct large underwater structures that can last for several hundred years, the use of HSC walls is highly recommended. Therefore, it is essential to understand overall behaviour of HSC walls since it may differ from normal strength concrete (NSC) walls. Despite its superiorities, HSC tends to be more brittle than NSC. Lessons learned from past experimental studies on NSC and HSC walls as mentioned previously can be used to understand overall behaviour of HSC walls, as the conclusions from NSC walls studies are also applicable to HSC walls. By doing so, it can help engineers to optimize the design and thus reducing the construction cost. However, currently there are very few experimental studies that investigated the behaviour of HSC walls with concrete strength of 100 MPa and above. In addition, there is also very

limited information in building codes that gives guidance to design HSC walls with such compressive strength. Hence, more experimental studies are needed to investigate further the behaviour of such HSC walls since they are the best representative of real structures as compared to numerical studies.

Recent analytical studies on structural walls had been done (Chandra et al. 2011; Chandra and Teng 2012) based on data collected from past experiments on RC walls. On these analytical studies, experimental wall strengths were compared with nominal wall strengths calculated from building codes, such as American code (ACI Committee 318 2014), Japanese code (Architectural Institute of Japan 1994), and European code (Comite Europeen de Normalisation 2004). From these studies, it was concluded that the flexure strength of RC walls could be predicted quite accurate using flexural theory for member subjected to axial load and bending moment. Moreover, most of building code formulas underestimated the shear strength of RC walls, especially HSC walls. In addition, the building code formulas to calculate the shear strength of RC walls were mostly empirical and the predictions were quite scattered with high values of coefficient of variation. Hence, there is a need to develop a new model based on rational theory to predict more accurately the shear strength of RC walls and verify it with experimental results on RC walls with wide ranging concrete strength from normal to high strength concrete.

1.2 Objectives and Scope

This study aims to investigate the shear behaviour of HSC walls with concrete compressive strength about 100 MPa. It is generally understood that shear behaviour is much more complicated than flexure behaviour. Moreover, from recent analytical studies by the author (Chandra et al. 2011; Chandra and Teng 2012), it can be concluded that the flexure strength of RC walls can be predicted reasonably accurate using a typical flexural theory. However, most building code formulas are not accurate enough to predict shear strength of RC walls, especially HSC walls.

The shear behaviour of HSC walls are investigated experimentally in the structural laboratory of Nanyang Technological University, Singapore. Several reduced-scale specimens are cast and tested until failure under axial loading and lateral cyclic loading that are typical loading conditions for RC walls. Furthermore, some significant factors that affect the shear strengths of RC walls, such as height to length ratio, vertical and horizontal web reinforcement ratios, and specimen shape, are also investigated in this experimental study.

Another objective of this study is to develop an analytical model based on the truss analogy to predict the shear strengths of RC walls. The main advantage of using the truss analogy is that flow of forces in the element or member with different span to depth ratios can be easily visualized by design engineers using compression struts and tension ties (ASCE-ACI Committee 445 1998). Moreover, by using the truss analogy, the influences of shear force and bending moment can be incorporated directly and simultaneously in the design. The proposed analytical model here is intended to serve as an accurate method that can be used directly by engineers to predict the shear strengths of RC walls. In addition, the proposed model is verified with a wide range of experimental results on RC walls from NSC to HSC to ensure its accuracy and applicability.

1.3 Thesis Layout

This study is presented in several chapters. It starts with “Introduction” part which explains the general background as well as the objectives and scope of this study. In the second chapter, literature review about past experiments on RC walls as well as several analytical models to predict the shear strength of RC walls that have been proposed by various researchers are discussed. Subsequently, an analytical study on the shear behaviour of RC walls is presented. In the fourth chapter, the experimental study conducted is explained in detail including the experimental results, discussions, and conclusions. In the fifth chapter, analytical models that have been proposed to predict the shear strength of RC walls are presented. In the last chapter, conclusions regarding this study as well as possible future works are presented.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

This chapter summarizes some key experimental studies on normal strength (below 60 MPa) and high strength (above 60 MPa) concrete walls from available literature. Furthermore, lessons learned from these experiments are also summarized and are used as a basis for planning this experimental study. In addition, some analytical models to calculate the shear strength of RC walls based on truss analogy are reviewed and discussed. Finally, building code provisions to calculate the shear strength of RC walls are also presented.

2.2 Normal Strength Concrete (NSC) Wall Experiments

As explained in the previous chapter, research on concrete walls began as early as in 1950s (Galletly 1952; Benjamin and Williams 1957; Hirose 1957). Since then, many experimental studies have been done by researchers to investigate further the behaviour of concrete walls. In this section, several key experimental studies on NSC walls with concrete strength below 60 MPa (Cardenas and Magura 1972; Barda et al. 1977; Cardenas et al. 1980; Corley et al. 1981; Lefas et al. 1990; Mo and Chan 1996; Salonikios et al. 1999; Salonikios et al. 2000; Chiou et al. 2003; Mosalam et al. 2003; Li et al. 2015; Tran and Wallace 2015) are discussed and summarized.

2.2.1 Cardenas and Magura (1972)

This research focused on investigating the strength of rectangular high rise shear walls subjected to static axial loading and static monotonic lateral loading. Six large scale specimens were cast with varying moment to shear ratio (shear span), amount and distribution of longitudinal reinforcement. Nominal concrete strength used was 42 MPa and nominal yield strength of steel bars was 420 MPa. Furthermore, moment to shear ratio were varied between 1.0 and 2.0 of the total wall length. Longitudinal reinforcement ranged between 0.27 to 3.00 per cent while

the amount of transverse reinforcement was kept constant (0.27 per cent). Moreover, the axial load ratio varied between 5.0 to 7.0 per cent of the total capacity of the concrete wall section. The overall experimental setup can be seen in Figure 2.1.

Figure 2.1 Specimen test setup (Cardenas and Magura 1972).

The conclusions drawn from this research are flexure behaviour is more dominant for rectangular high rise shear walls rather than shear behaviour and the flexural strength of the walls can be calculated using same assumptions as for beams.

2.2.2 Barda, Hanson, and Corley (1977)

In this experiment, eight low rise walls with boundary elements were tested to obtain data on strength, energy absorption, and effect of cyclic lateral loading. Parameters varied in this test were height to length ratio and vertical and horizontal reinforcement ratios. Concrete compressive strength used varied between 16 to 29 MPa and nominal yield strength of steel bars was 420 MPa. The vertical reinforcement in the boundary elements was varied between 1.8 to 6.4 per cent of the area of the

- Shear strength of walls having height to length ratio of 1.00 is about 20 per cent lower as compared to similar specimens having height to length ratio of 0.25 and 0.50.

2.2.3 Cardenas, Russell, and Corley (1980)

In this study, strength of rectangular low rise walls was investigated by testing seven large scale specimens having height to length ratio of 1.0 subjected to static lateral monotonic loading without applying axial load. Only one specimen was subjected to cyclic lateral loading. Variables in this experiment were amount and distribution of vertical and horizontal reinforcement. The vertical reinforcement was varied between 0.30 to 3.00 per cent while the horizontal reinforcement ranged from 0.27 to 1.00 per cent. Nominal concrete compressive strength was 41.4 MPa and nominal yield strength of steel bars was 420 MPa. Overall dimensions of the specimens can be seen in Figure 2.3.

Figure 2.3 Dimensions of test specimens (Cardenas et al. 1980).

Conclusions drawn from this study are rectangular low rise walls can develop shear stresses up to $0.83 \sqrt{f'_c}$ MPa and shear strength of walls

subjected to monotonic loading is seven per cent higher than similar one subjected to cyclic loading. In addition, for walls with height to length ratio of 1.0, it was found that both vertical and horizontal reinforcement contribute effectively to the shear strength of walls.

2.2.4 Corley, Fiorato, and Oesterle (1981)

This research focused on investigating effects of loading history, walls sectional shape, vertical and horizontal reinforcement ratio, axial load ratio, and concrete compressive strength to the strength and deformation capacity of structural walls. Nineteen specimens were cast with several different wall section shapes (rectangular, with boundary columns, with flanges) as seen in Figure 2.4b. Concrete compressive strength varied between 22 to 54 MPa while yield strength of steel bars ranged between 410 to 540 MPa. Axial load ratio was varied between 0.00 to 0.14 per cent of the total capacity of the concrete wall section. Furthermore, vertical reinforcement in boundary elements ranged between 1.11 to 4.35 per cent while vertical reinforcement in walls was varied between 0.20 to 0.31 per cent. For horizontal reinforcement in walls, it ranged between 0.31 to 1.38 per cent. Nominal dimensions of test specimen can be seen further in Figure 2.4.

Several key findings from this research are:

- Strength of walls taken as minimum between flexural and shear strength calculated using ACI code formula (ACI Committee 318 1971) are lower than the actual ones and the predicted failure modes are not necessarily the same as the actual ones.
- For walls governed by flexural failure, application of cyclic loading results in 15 per cent reduction in strength whereas for ones governed by web crushing (shear) failure, the effect is not as significant.
- Walls with boundary elements or flanges have higher flexural and shear strength as compared to similar rectangular ones. Furthermore, rectangular walls are more susceptible to lateral instability during cyclic loading.

- Providing excessive horizontal reinforcement beyond ACI code (ACI Committee 318 1971) requirement does not significantly increase the shear strength of walls since the web crushing failure mode will govern.
- Presence of axial loading can increase flexural and shear strength of walls.
- Walls with lower concrete strength have less deformation capacity as compared to those with higher concrete strength.

Figure 2.4 Test specimens (Corley et al. 1981).

2.2.5 Lefas, Kotsovos, and Ambraseys (1990)

In this experiment, thirteen rectangular wall specimens were tested under axial loading and static lateral monotonic loading with varying height to length ratio, axial load ratio, concrete strength, and horizontal reinforcement ratio. The aim of this experiment was to investigate parameters affecting walls behaviour. Height to length ratio used were 1.0 and 2.0. Moreover, concrete strength was varied between 25 to 45 MPa and yield strength of steel bars ranged between 420 to 520 MPa. Axial load ratios used were 0.0, 0.1, and 0.2 of the total capacity of the concrete wall section. Vertical reinforcement used was around 2.4 or 2.5 per cent while horizontal reinforcement varied between 0.4 to 1.1 per cent. Specimen details can be seen further in Figure 2.5.

From this experiment, it can be concluded that axial load increases strength capacity and stiffness of walls yet reduces ductility. This finding becomes more remarkable as height to length ratio increases. Moreover, strength and deformation capacity of walls seem to be independent of concrete strength for f'_c within range of 25 to 45 MPa.

Figure 2.5 Specimen details (Lefas et al. 1990).

2.2.6 Mo and Chan (1996)

This study aimed to investigate the effect of loading rate on the behaviour of low rise shear walls and to examine whether truss model theory as proposed by Hsu and Mo (1985) can be applied to low rise shear walls with concrete strength up to 60 MPa. In this study, twenty low rise framed shear walls were tested with varying parameters such as concrete compressive strength, steel yield stress, vertical reinforcement ratio, and cyclic loading hysteresis. Concrete compressive strength was varied between 18 to 66 MPa while steel yield stress was varied between 302 to 443 MPa. Two ratios of vertical reinforcement were used, i.e. 0.52% and 0.65%. Three different loading histories were used. The first type, the magnitude of loading was increased with increment in number of cycles while in the second type, it was the opposite. The third type was combination of the first and the second type of loadings. Dimensions of test specimen can be seen in Figure 2.6.

Figure 2.6 Dimensions of test specimen (Mo and Chan 1996).

From this study, it was observed that the calculated ultimate shear forces using truss model theory (Hsu and Mo 1985) were in average

higher than the experimental shear forces for walls subjected to dynamic shear force with low frequency (0.2 Hz). Furthermore, the effect of cyclic loading history was found to be significant. Maximum shear forces recorded from reversed cyclic load were less than those recorded from monotonic load. This was due to transverse tensile strains and diagonal cracks reduced the strength of diagonal concrete struts.

2.2.7 Salonikios, Kappos, Tegos, and Penelis (1999; 2000)

This study aimed to investigate the behaviour of squat rectangular concrete walls subjected to cyclic lateral loading and to assess the validity of design provisions provided by Eurocode 8 (Comite Europeen de Normalisation 1995) and ACI code 318 (ACI Committee 318 1995). It involved testing of total eleven specimens with height to length ratio of 1.0 and 1.5 designed according to Eurocode 8 (Comite Europeen de Normalisation 1995) with varying axial load ratio and web reinforcement ratio. Concrete strength varied from 22 to 27 MPa while the yield strengths of steel bars were 585 and 610 MPa for vertical reinforcement in boundary elements and web reinforcement, respectively. Axial load ratios used were 0.00 and 0.07 of the total capacity of the concrete wall section. Furthermore, vertical reinforcement ratios in boundary elements were 1.3 and 1.7 per cent while vertical and horizontal reinforcement in walls were 0.28 and 0.56 per cent.

From this study, it was learned that despite of having low height to length ratio (1.0 and 1.5), almost all walls could fail in flexure mode and had sufficient energy dissipation capacity. In addition, from the results, it can be seen that shear strength of walls calculated by ACI code formula are higher than their flexural strength obtained from analysis which means ACI code correctly predicted the failure mode of the walls tested. On the other hand, for Eurocode, the calculated shear strength of some walls are lower than their flexural strength obtained from analysis which means Eurocode underestimated the shear capacity of some walls since in the reality, almost all walls, with only one exception, failed in flexure

mode. Overall, shear capacity of walls calculated from ACI code are mostly higher than those calculated from Eurocode.

2.2.8 Chiou, Mo, Hsiao, Liou, and Sheu (2003)

In this study, behaviour of ten large scale wall specimens, including high, mid, and low rise walls subjected to cyclic lateral loading without axial load was investigated. The parameters varied were concrete strength and vertical reinforcement ratio. For high rise walls, the height to length ratio was 2.00 while for mid and low rise walls, the ratios were 1.00 and 0.75, respectively. Concrete strength was varied between 22 to 28 MPa and steel bars yield strength ranged between 428 to 458 MPa. Vertical reinforcement ratios used were 0.27 and 0.37 per cent while horizontal reinforcement ratio was kept constant at value of 0.27 per cent.

Several findings from this study are:

- High rise walls tend to fail in flexure mode and their ductility is higher as compared to low rise walls.
- Mid rise walls tend to fail in combination of flexure and shear mode while low rise walls tend to fail in shear mode.
- Specimens with higher concrete strength and higher vertical reinforcement ratio exhibit greater maximum loads.
- Contribution of vertical reinforcement to the strength of walls is more significant for specimens failing in flexure rather than for those failing in shear.
- Prediction formula developed previously from small scale specimens overestimates the strength of walls for large scale specimens. It implies that size of specimens significantly affects structural behaviour of walls and the effect needs to be considered in future studies.

2.2.9 Mosalam, Mahin, and Rojansky (2003)

This study investigated the behaviour of lightweight reinforced concrete shear walls with embedded structural steel members, especially the brittle in-plane splitting failure caused by knife edge loading of

embedded steel members. Furthermore, in this study, several retrofit strategies were also proposed and evaluated. There were a total of fourteen specimens tested. The parameters varied were wall reinforcement, confining rods, and loading shoes insert (to simulate different knife-edge loading conditions on concrete shear walls). Concrete compressive strength used was 36.8 MPa while steel bars with minimum yield strength of 414 MPa were used as the reinforcing steel for wall panels. The test specimen and loading conditions can be seen in Figure 2.7.

Figure 2.7 Test specimen and loading conditions (Mosalam et al. 2003).

From the study, it was concluded that wall reinforcement ratio was not a significant parameter to the behaviour of lightweight reinforced concrete shear walls. Furthermore, through-thickness confinement rods (bonded or unbonded) were able to improve significantly the performance of lightweight reinforced concrete shear walls. In addition, smaller width of knife-edge loading might lead to more brittle failure since it increased the probability of lightweight reinforced concrete shear walls to split.

2.2.10 Li, Pan, and Xiang (2015)

This study aimed to evaluate the seismic performance of RC squat walls with less confining reinforcement than that recommended by ACI 318 (2008). There were eight specimens tested with varying parameters of wall height to length ratio, axial load ratio, transverse reinforcement

ratio in wall boundary elements, and presence of construction joints at wall base. Concrete compressive strength was varied between 34.8 to 41.6 MPa and steel yield strength varied from 261 to 308 MPa (for round bars) and 427 to 519 MPa (for deformed bars). Vertical and horizontal web reinforcement ratio were kept constant at 0.50% for all specimens. The details of specimen geometry and reinforcement can be seen in Figure 2.8. Further details of this study can be found in Xiang (2009).

Figure 2.8 Details of specimen geometry and reinforcement (Li et al. 2015).

From this study, it was found that all specimens failed in flexure and they exhibited more ductile behaviour than expected although they had less transverse reinforcement in boundary elements than that required by ACI 318 (2008). Nevertheless, it was found that increasing amount of transverse reinforcement in boundary elements could make contribution of flexural deformation to total wall deformation become larger and thus increasing drift and energy dissipation capacity of RC squat walls. Furthermore, it was found that axial load ratio was not significant to the degradation rate of secant stiffness. However, specimens with higher axial load ratio had better energy dissipation capacity. In addition,

presence of construction joints had negligible effect on stiffness characteristics of RC squat walls.

2.2.11 Tran and Wallace (2015)

This study aimed to investigate nonlinear cyclic behaviour of moderate aspect ratio (with H_w/L_w of 1.5 to 2.0) RC structural walls. The specimens were designed to have flexural yielding before reaching their nominal shear strength according to ACI 318 (2011). There were a total of five specimens tested with varying parameters of wall height to length ratio, axial load ratio, and wall shear stress level. Concrete compressive strength was varied between 47.1 to 57.5 MPa (at the day of testing) while steel bars yield strength was varied between 423 to 516 MPa. Vertical and horizontal web reinforcement were set to satisfy or exceed the minimum requirement by ACI 318 (2011). The details of specimen dimensions and reinforcement can be seen in Figure 2.9.

Figure 2.9 Details of specimen dimensions and reinforcement (Tran and Wallace 2015).

From this study, it was found that first yielding of flexural reinforcement in boundary elements had been observed at drift level of 0.5 to 0.7% for all specimens while the specimens themselves failed at drift level of approximately 3.0%. The mode of failures observed were diagonal tension failure, web crushing, concrete crushing, horizontal crushed concrete zone sliding, and buckling of boundary and web vertical reinforcement. In addition, it was found that contribution of shear deformation to total wall deformation was varied between 20 to 50% for walls with height to length ratio of 2.0 and 1.5, respectively. The data from this study will be used for development and validation of analytical models that account for nonlinear shear-flexure interaction.

2.3 High Strength Concrete (HSC) Wall Experiments

In this section, several key experiments on HSC walls with concrete strength above 60 MPa (Gupta and Rangan 1998; Kabeyasawa and Hiraishi 1998; Yun et al. 2004; Farvashany et al. 2008; Yan et al. 2008; Liang et al. 2013; Burgueno et al. 2014) are summarized and discussed.

2.3.1 Gupta and Rangan (1998)

In this study, behaviour of eight one-third scale HSC wall specimens under axial loading and static monotonic lateral loading was investigated. Parameters studied were axial load ratio and vertical and horizontal reinforcement ratio. Only one specimen was designed to fail in flexure and the others were designed to fail in shear. Furthermore, another objective was to compare the actual strength of HSC walls with nominal strength obtained from building codes (Standards Australia 1988; ACI Committee 318 1995). Nominal concrete strength used was 70 MPa and steel bars yield strength varied from 529 to 578 MPa. Height to length ratio was kept constant at value of 1.00 while axial load ratio ranged from 0.00 to 0.13 of the total capacity of the concrete wall section. Vertical reinforcement ratios selected were 1.0 and 1.5 percent of the wall cross

section area while horizontal reinforcement ratios were 0.5 and 1.0 per cent. Dimensions of specimens can be seen in Figure 2.10.

From the test results, it can be concluded that:

- Axial load increases the shear strength of HSC walls but reduces deformation capacity.
- Both vertical and horizontal reinforcement are effective in contributing to the shear strength of HSC walls with height to length ratio of 1.00.
- Australian code (Standards Australia 1988) and ACI code (ACI Committee 318 1995) significantly underestimate the shear strength of HSC walls.

Figure 2.10 Dimensions of wall specimens (Gupta and Rangan 1998).

2.3.2 Kabeyasawa and Hiraishi (1998)

In this research, total of twenty one one-quarter scaled HSC wall specimens were tested to investigate the hysteretic behaviour (including flexure and shear strength) of HSC walls and to develop a general design method based on resistance mechanisms. Moreover, another objective was to review and summarize other experiments of HSC walls with concrete strength more than 60 MPa that had been conducted in Japan

(Maeda 1986; Kumagai 1990; Okamoto 1990; Sugano 1991; Nagashima 1993). Parameters studied in this research were loading conditions and reinforcement ratios. Concrete strength used varied from 60 to 120 MPa and steel bars yield strength ranged from 700 to 1200 MPa. Shear span ratio was varied from 0.60 to 2.00 of total wall length while axial load ratio ranged from 0.00 to 0.15 of wall gross section area. Vertical as well as horizontal reinforcement ratios were varied from 0.27 to 2.00 per cent. Dimensions of sample specimen can be seen in Figure 2.11.

Figure 2.11 Dimensions of sample specimen (Kabeyasawa and Hiraishi 1998).

Several conclusions are drawn from this research:

- HSC walls can exhibit ductile flexural failure. However, observed pinching hysteretic behaviour may be caused by limited inelastic

deformation of high strength steel bars used and high axial load ratio applied.

- The flexure strength of HSC walls can be predicted quite well using flexural theory.
- Shear design equation of Japanese code (Architectural Institute of Japan 1994) with modification in the effective factor for high strength concrete gives fair estimation of the strength of HSC walls failing in shear with yielding of web reinforcement. However, the equation overestimates the strength of HSC walls failing in shear without yielding of web reinforcement.
- Shear strength of highly web reinforced specimens are not much higher than those expected by truss analogy. This implies that putting excessive web reinforcement more than required does not increase much the shear strength of HSC walls since the failure will be governed by web crushing.

2.3.3 Yun, Choi, and Lee (2004)

In this experiment, five one-third scaled HSC flexural walls were tested under axial loading and cyclic lateral loading to investigate the behaviour and strength of HSC flexural walls with varying axial load ratio and horizontal reinforcement ratio. Moreover, it was also intended to verify design provisions provided by ACI code (ACI Committee 318 1999) and AIJ guideline (Architectural Institute of Japan 1990). Nominal concrete strength used was 69 MPa while yield strength of steel bars was 414 MPa. Shear span ratio was kept constant at value of 1.80 while axial load ratio was varied from 0.00 to 0.06 of the total capacity of the concrete wall section. Vertical reinforcement ratio was kept constant at value of 0.55 per cent whereas horizontal reinforcement ratio ranged from 0.28 to 1.10 per cent. Details of specimens can be seen in Figure 2.12.

From this experiment, several lessons can be learned:

- It is possible to achieve ductile behaviour of flexural walls using high strength concrete.

- Specimens subjected to higher axial load have higher strength but less ductility and they exhibit early degradation of strength and stiffness.
- Horizontal reinforcement has little effect to the strength and deformation characteristics of HSC walls failing in flexure.
- ACI code (ACI Committee 318 1999) and AIJ guideline (Architectural Institute of Japan 1990) slightly underestimate the flexure strength of HSC walls.

Figure 2.12 Details of specimens (Yun et al. 2004).

2.3.4 Farvashany, Foster, and Rangan (2008)

In this research, shear behaviour of HSC walls was investigated by testing seven one-third scale specimens under axial loading and static monotonic lateral loading. Parameters studied were axial load ratio and web reinforcement ratios. Concrete strength used was varied from 80 to 100 MPa. Height to length ratio was kept constant at value of 1.25 while axial load ratio ranged from 0.04 to 0.22 of the total capacity of the

concrete wall section. Vertical reinforcement ratios were 0.75 and 1.26 per cent while horizontal reinforcement ratios were 0.47 and 0.75 per cent. Details of specimens can be seen in Figure 2.13.

Figure 2.13 Details of specimens (Farvashany et al. 2008).

From this research, it can be learned that:

- HSC walls with higher axial load exhibit greater shear strength but less ductility.
- Specimens with higher amount of horizontal reinforcement have higher ultimate displacement. This means that horizontal reinforcement has important effect on the ductility of HSC walls. However, the effect of horizontal reinforcement is not that significant on the shear strength of HSC walls. Specimens with higher amount of horizontal reinforcement do not exhibit much greater shear strength.

- Shear strength formulas given by Australian standard (AS3600 2001) and ACI code (ACI Committee 318 2002) significantly underestimate the actual shear strength of HSC walls with large discrepancy.

2.3.5 Yan, Zhang, and Zhang (2008)

This experiment developed a strategy to improve the capacity and deformation of HSC walls using high strength steel bars. Five rectangular HSC wall specimens were tested under axial loading and cyclic lateral loading with varying steel bars yield strength, concrete strength, axial load ratio, and shear span ratio. Concrete strength used was varied from 60 to 90 MPa while steel bars yield strength ranged from 235 to 1240 MPa. Shear span ratios were 1.50 and 2.36 of total wall length while axial load ratios were 0.15 and 0.25 of the total capacity of the concrete wall section. Vertical and horizontal reinforcement ratios were 0.71 and 0.85 per cent, respectively.

From this experiment, conclusions drawn are:

- Capacity and ductility of HSC walls can be increased using high strength bars as stirrups in boundary elements and as horizontal reinforcement in the web.
- Specimen with lower concrete strength has lower capacity but more ductility.
- Axial load increases strength of HSC walls but reduces ductility.
- HSC wall with lower shear span ratio exhibits greater strength but less ductility.

2.3.6 Liang, Che, Yang, and Deng (2013)

In this study, the performance of six HSC walls with edge columns and high strength stirrup chains in boundary elements under cyclic lateral loading was investigated. Moreover, the effects of axial load ratio, height to length ratio, and amount of confinement reinforcement on ductility, energy dissipation capacity, and failure pattern of HSC walls were observed as well. Concrete cube compressive strength was about 93.8

MPa at the day of testing and steel bars yield strength was varied between 419 to 494 MPa. Height to length ratio of the walls ranged from 1.0 to 2.1 and axial load ratios were 0.21 and 0.28. All specimens were designed with sufficient horizontal reinforcement so that the shear capacity of the walls was greater than the flexural capacity. Dimensions and reinforcement arrangement for the specimens can be seen in Figure 2.14.

(a) DHSCW-01, DHSCW-03, DHSCW-05

(b) DHSCW-02, DHSCW-04, DHSCW-06

Figure 2.14 Wall reinforcement details (Liang et al. 2013).

Several conclusions can be drawn from this study:

- The deformation and energy dissipation capacity of HSC walls with edge columns and high strength stirrup chains were significantly improved. This was due to the edge columns (increased the cross sectional area of the walls) and increase of amount and effectiveness of confining stirrups. The test results showed that despite of high axial load ratio used (0.28), HSC walls could still obtain high ductility ratio.

- Flexural yielding was observed in all specimens prior to failure with high rise walls experienced flexural failure whereas medium and low rise walls exhibited shear failure. Thus, deformation and energy dissipation capacity of high rise walls were better than those of medium and low rise walls. However, lateral load carrying capacity of high rise walls was less than that of medium and low rise walls.

2.3.7 Burgueno, Liu, and Hines (2014)

This study aimed to investigate the correlation between concrete compressive strength and web crushing capacity of RC structural walls with boundary elements. There were two hypotheses to be confirmed. First, web crushing capacity increases proportionally with concrete compressive strength as long as diagonal concrete struts are not damaged. Second, damaged diagonal concrete struts due to cyclic loading and inelastic deformation can limit web crushing capacity independently of concrete compressive strength. Hence, in order to examine these two hypotheses, there were a total of eight specimens tested under monotonic and cyclic loading with concrete compressive strength ranging from 39 to 138 MPa. Parameters varied were concrete compressive strength and loading type. Height to length ratio of all specimens was 2.5. Vertical and horizontal web reinforcement ratio used were about 1.5% and 2.0%, respectively. Cross sectional details of specimens can be seen in Figure 2.15.

From the results of this study, two hypotheses mentioned before were confirmed. Furthermore, it was concluded that increase in concrete compressive strength could improve ductility and energy dissipation capacity of RC walls by preventing web crushing shear failures. Moreover, increase in concrete compressive strength did not linearly correlate with increase in web crushing capacity. Instead, the correlation was nonlinear, with a decreasing limit as concrete compressive strength increases. In addition, RC walls with well-confined boundary elements

exhibited web crushing capacity greater than acceptable level for wide range of concrete compressive strengths.

Figure 2.15 Cross sectional details of specimens (Burgueno et al. 2014).

2.4 Lessons Learned from Past Experiments on Concrete Walls

In this section, lessons learned from previous research regarding overall behaviour of concrete walls are presented. Several key parameters that affect the behaviour of concrete walls are height to length ratio, axial load ratio, vertical and horizontal reinforcement ratio, concrete strength, loading history, and wall shape and size. The influences of each parameter on concrete walls behaviour are discussed in the following subsections.

2.4.1 Height to Length Ratio

Height to length ratio (aspect ratio) is also well known as shear span ratio (moment to shear ratio) for cantilever walls. However, this definition may not be correct for double curvature walls. Thus, this

parameter has to be defined carefully for each case (experimental setup or wall configuration) in order to obtain the correct value.

As in beams, shear span ratio or height to length ratio for walls plays important roles that affect the overall behaviour. Slender wall with high height to length ratio (≥ 2.0) tends to be dominated by flexure behaviour rather than shear whereas squat wall with low height to length ratio (≤ 1.0) tends to be dominated by shear behaviour. Moreover, for wall with height to length ratio between 1.0 and 2.0, the behaviour is a combination of flexure and shear.

Previous experiments (Barda et al. 1977; Yan et al. 2008; Liang et al. 2013) have shown that walls with lower height to length ratio (whether fail in flexure or shear) have higher strength than those with higher height to length ratio. For flexural walls having exactly the same cross section and reinforcement properties, lower height to length ratio simply means less moment arm and thus it needs higher lateral load to achieve the maximum moment capacity as compared to walls with higher height to length ratio. For walls failing in shear, higher height to length ratio may cause cracks to occur at earlier loading stage and hence it may reduce the concrete contribution to the shear strength of walls.

In addition, height to length ratio also has effect on the ductility of walls. Walls with low height to length ratio have lower ductility or deformation capacity as compared to walls with high height to length ratio. This behaviour can be related to the type of failure mechanism. Walls with high height to length ratio tend to fail in flexure which is a ductile failure whereas walls with low height to length ratio tend to fail in shear which is a brittle failure.

2.4.2 Axial Load Ratio

Axial load ratio is defined as ratio of applied axial loading to the wall gross section area multiplied by concrete compressive strength. Effect of axial loading on concrete walls is found to be similar with that on concrete columns.

Past experiments on NSC walls (Corley et al. 1981; Lefas et al. 1990) and HSC walls (Gupta and Rangan 1998; Yun et al. 2004; Farvashany et al. 2008; Yan et al. 2008) concluded that presence of axial loading could increase the flexural and shear strength of walls but reduce the ductility. Moreover, rectangular RC walls with very high axial load ratio (>0.25) tend to fail in out of plane buckling mode after post-yielding stage. However, RC walls with well-confined boundary elements are able to reach high ductility ratio even though very high axial load ratio (0.28) is applied (Liang et al. 2013).

2.4.3 Vertical and Horizontal Reinforcement Ratio

Here, vertical reinforcement refers to vertical web reinforcement in walls while horizontal reinforcement refers to horizontal web reinforcement in walls. In fact, many research (Barda et al. 1977; Cardenas et al. 1980; Corley et al. 1981; Gupta and Rangan 1998; Kabeyasawa and Hiraishi 1998; Chiou et al. 2003; Yun et al. 2004; Farvashany et al. 2008; Yan et al. 2008) have been conducted to investigate the effects of vertical and horizontal reinforcement to the behaviour of concrete walls. The findings are summarized in the following paragraphs.

For walls failing in flexure, amount and distribution of vertical reinforcement significantly affect the flexure strength of walls while for horizontal reinforcement, the effect is not significant. However, horizontal reinforcement is still important to provide confinement for the web to ensure sufficient ductility of the walls.

For walls failing in shear, the effects of web reinforcement are more complex and vary with height to length ratio of the walls. For walls with height to length ratio of 1.0, both vertical and horizontal reinforcement are effectively contributing to the shear strength of walls. For walls with height to length ratio less than 1.0, vertical reinforcement seems to be more effective than horizontal reinforcement whereas the opposite is true for walls with height to length ratio more than 1.0. Furthermore,

providing horizontal reinforcement more than required will not increase much the shear strength of walls since the failure will be governed by web crushing mode. For the effect to the ductility of walls, higher amount of horizontal reinforcement can give more confinement and thus results in higher ductility.

2.4.4 Concrete Strength

Despite having higher strength and higher stiffness, it is well known that high strength concrete is more brittle and has less ductility as compared to normal strength concrete. Hence, it may limit the application of high strength concrete in structural systems (in this case shear walls) which are designed to resist lateral loads generated from earthquake since the loadings usually exceed the elastic limit of structures.

However, recent studies on HSC walls (Kabeyasawa and Hiraishi 1998; Yun et al. 2004) show that it is possible to have HSC flexural walls that are able to fail in ductile manner. It is suggested that axial load ratio in the walls should be kept low in order to sustain the ductility. Furthermore, the usage of ultra-high strength steel bars with yield strength more than 700 MPa should be avoided to prevent pinching hysteretic behaviour due to small inelastic deformation of the steel bars.

Another study on HSC walls (Burgueno et al. 2014) shows that increasing concrete compressive strength could increase web crushing capacity of RC walls although the correlation is not linear. Furthermore, they also concluded that well-confined boundary elements helped to increase web crushing capacity for wide ranges of concrete compressive strengths. Therefore, from these studies, it can be concluded that it is reasonable to utilize the benefits of HSC walls in terms of higher strength and higher stiffness as long as proper design method and detailing for ductility are maintained.

2.4.5 Loading History

Loading history here means how the lateral loading is applied to the walls. Previous experiments for walls usually use two methods for applying the lateral loading. They are static monotonic loading (one directional loading increased gradually until failure) and static cyclic loading (reversal loading with increasing magnitude until failure). Previous studies (Barda et al. 1977; Cardenas et al. 1980; Corley et al. 1981; Mo and Chan 1996) show that application of static cyclic loading results in reduction of strength (whether fail in flexure or shear) as compared to identical specimens subjected to static monotonic loading. For walls failing in shear, the reduction in strength is about 7 to 10 per cent whereas for those failing in flexure, it is about 15 per cent. According to experimental results conducted by Corley et al. (1981), this is because for walls failing in flexure, the capacity of vertical reinforcement is greatly affected by inelastic stress reversals and occurrence of local buckling. On the other hand, for walls failing in shear, the mechanism of load resistance depends on diagonal struts that do not deteriorate significantly under load reversals. Thus, the effect of inelastic stress reversals on the vertical reinforcement capacity is not as significant.

2.4.6 Wall Shape and Size

Based on the shape, concrete structural walls can be classified into two types, i.e. rectangular walls and walls with boundary elements. The boundary elements may take form of columns or flanges as can be seen in Figure 2.4b. Previous study (Corley et al. 1981) shows that walls with boundary elements can exhibit greater strength as compared to rectangular walls. The reasons for this are explained in the following paragraphs.

Walls with boundary elements provide more area for vertical reinforcement to be placed at both ends as compared to rectangular walls. Thus, greater amount of concentrated vertical reinforcement at both ends of walls can significantly increase the flexural capacity. Furthermore,

these boundary elements can act as large dowels which are able to prevent sliding shear failure and provide residual capacity in case of web crushing. Walls with boundary elements usually have higher shear stress at web (i.e. higher shear strength) as compared to rectangular walls and the failure mode will be governed by web crushing.

Another study (Liang et al. 2013) found that the deformation and energy dissipation capacity of HSC walls with edge columns and high strength stirrup chains are significantly improved. This is due to the edge columns (increased the cross sectional area of the walls) and increment of amount and scope of confining stirrups. Moreover, another benefit worth noting is that RC walls with boundary elements have higher in and out of plane stiffness and hence they are more stable against axial and lateral loadings. The test results show that despite of high axial load ratio used (0.28), HSC walls could still obtain high ductility ratio. On the other hand, rectangular RC walls are more prone to lateral instability during cyclic loading.

For size effect, previous study (Chiou et al. 2003) concluded that size effect has to be considered in the analytical model of concrete walls since it affects the structural behaviour of the walls. The prediction formula once developed for small scale wall specimens overestimates the strength of large scale wall specimens. Therefore, it is suggested for future studies that larger scale tests are needed to investigate further the size effect in concrete structural walls.

2.5 Analytical Models for Predicting Shear Strength of RC Walls

In this section, state of the art of analytical models for predicting shear strength of RC walls is reviewed. Some major analytical models are discussed in details including their basic assumptions, their mathematical formulations, and their approaches to predict the shear strength of RC walls. Moreover, at the end of this section, comments regarding each model are briefly summarized.

As widely known, unlike flexure behaviour which is well-understood, shear behaviour of RC members is complex in nature (Park and Paulay 1975). There

are many factors that affect shear strength of RC members and some of these factors are not easily to be measured, such as aggregate interlock and dowel action. The current building code formulas are still based on empirical equations to calculate the shear strength of RC members and their formulations are very different each other (Hsu 1988).

In a report by joint committee between American Society of Civil Engineers (ASCE) and American Concrete Institute (ACI) (ASCE-ACI Committee 445 1998), it is stated that recent analytical approaches for shear design of structural concrete are based on truss analogy. This theory was originally developed in early 1900s (Ritter 1899; Morsch 1909) and since then it has undergone many major developments to obtain better accuracy in predicting the shear strength of RC members.

Starting from 1960s, the development of the truss analogy for shear began to set its foundation on two-dimensional RC membrane elements which satisfy three basic principles of mechanics of materials (see Hsu (1998)). These principles are equilibrium, compatibility, and constitutive law of cracked reinforced concrete. This development resulted into three rational truss models, i.e. equilibrium (plasticity) truss model (Nielsen 1967; Lampert and Thurlimann 1968), Mohr compatibility truss model (Baumann 1972; Collins 1973; Collins and Mitchell 1980), and softened truss model (Hsu 1988; Hsu 1993; Pang and Hsu 1996).

Equilibrium (plasticity) truss model assumes that two-dimensional equilibrium condition is achieved within the RC membrane elements subjected to shear and normal stresses (Nielsen 1967; Lampert and Thurlimann 1968). Furthermore, longitudinal and transverse reinforcements are assumed to yield at failure whereas the concrete tensile stress is neglected. In this model, the shear strength of the RC membrane elements can be calculated from the product of geometric mean of web reinforcement contributions ($\sqrt{[\rho_l f_{yl} \rho_t f_{yt}]}$) for the case of pure shear without normal stresses. However, the shear strength of the membrane elements can only be achieved at ultimate load stage by using this model.

Mohr compatibility truss model assumes that two-dimensional compatibility condition is satisfied within the RC membrane elements subjected to shear and normal stresses (Baumann 1972; Collins 1973). In this model, simple linear stress-strain relationships for concrete and steel based on Hooke's law are used. In addition, the tensile strength of concrete is also neglected as in the case of equilibrium (plasticity) truss model. This compatibility truss model is able to satisfy two-dimensional forces equilibrium, Mohr's compatibility equations, and Hooke's linear material constitutive law. However, it is applicable only for linear analysis (Hsu 1996).

Softened truss model emphasizes the importance of taking into account the softening behaviour of concrete. This softening behaviour of concrete was first discovered by Robinson and Demorieux (1968; 1972). It was found that for two-dimensional elements, the principal tensile stress in the perpendicular direction caused a reduction or softening in the principal compressive stress. Later, Vecchio and Collins (1981) found that the softening coefficient was a function of principal tensile strain rather than principal tensile stress. The combination of softened constitutive laws for concrete, two-dimensional forces equilibrium, and Mohr's compatibility equations serves as the basis for the softened truss model or the modified compression field theory. The advantage of this model as compared to two previous models is that this model is applicable for service load stage as well as ultimate load stage.

These two dimensional models utilizing membrane elements are suitable for main regions of RC members where stresses and strains are distributed uniformly. Nevertheless, they may not be suitable for local regions or disturbed regions where stresses and strains are irregular, and thus compatibility conditions are difficult to be satisfied. In local regions or disturbed regions such as beam-column joints, corbels, beam or column ends, regions near concentrated load, etc., strut and tie model, instead, may serve as a suitable analytical model for shear design of RC members.

As in the case of truss model, the strut and tie model is also based on truss analogy. In this theory, a RC member is assumed to be comprised of concrete struts which resist compression forces and steel ties which resist tension forces.

The struts and ties together form a truss and thus is called “truss analogy”. However, to differentiate between these two models, hence, the model with regular angle of concrete struts and is applicable to main regions is called “truss model” whereas the model with irregular angle of concrete struts and is applicable to local or disturbed regions is called “strut and tie model” (Hsu 1996).

A special report by Schlaich et al. (1987) provides a clear explanation of strut and tie model and it gives design examples of disturbed regions utilizing strut and tie model. This model has served as a rational approach for designing disturbed regions by replacing the so-called “good practice” or “past experience” (Schlaich et al. 1987). Moreover, strut and tie model has also been included in several building codes, such as ACI 318 (ACI Committee 318 2002) and Eurocode 2 (Comite Europeen de Normalisation 2004), as an alternative method for designing local or disturbed regions.

Currently, for RC walls, there are two major analytical models based on truss analogy that have been developed to predict the strength of walls failing in shear. They are softened truss model (Hsu and Mo 1985; Mau and Hsu 1986) and softened strut and tie model (Hwang et al. 2001; Hwang and Lee 2002). The main advantage of these two models is their capability to incorporate three basic principles of mechanic of materials, i.e. equilibrium, compatibility, and constitutive laws of cracked concrete. Nevertheless, there is one fundamental difference in these two models on the assumption of stresses and strains distribution in the wall panel. The softened truss model assumes that stresses and strains are distributed uniformly throughout the wall panel while the softened strut and tie model assumes that stresses and strains are concentrated mainly in the compressive struts and tension ties which form a strut and tie model within the wall panel. These assumptions differentiate their mathematical formulations and approach in predicting the shear strength of RC walls. Further details of these two models are discussed in following sections.

2.5.1 Softened Truss Model and Compression Field Theory

One of the earliest adoptions of softened truss model in predicting shear strength of RC walls was performed by Hsu and Mo (1985). Later, it was followed by Mau and Hsu (1986) to simplify the procedure in the preceding work for design purposes. After these two pioneer works, many research have been done to improve the softened truss model for calculating the shear strength of RC walls (Mau and Hsu 1987; Gupta and Rangan 1998; Mansour et al. 2004; Kassem and Elsheikh 2010). The basis and development of this model are explained in the following paragraphs.

As explained before, the word “softened” emphasizes the importance of taking into account softening behaviour of concrete (Hsu 1988). This phenomenon was first discovered by Robinson and Demorieux (1968; 1972) and later was quantified by Vecchio and Collins (1981). The discovery of softening behaviour of concrete gave rise to the establishment of the so-called “compression field theory” (Collins and Mitchell 1980). This theory incorporates three basic principles of mechanics of materials, i.e. equilibrium, compatibility, and softened constitutive laws of concrete. However, this theory assumes that there will not be tensile stresses in concrete after cracking.

The compression field theory (CFT) served as a basis for the softened truss model. As mentioned before, one of the earliest implementations of the softened truss model to calculate the shear strength of RC walls was done by Hsu and Mo (1985). In their paper, they derived the truss model theory to predict the strength and behaviour of low rise RC shear walls. In this theory, three basic principles of mechanics of materials were incorporated. The equilibrium conditions can be viewed in Figure 2.16 and equations (2.1) to (2.3) while the compatibility conditions are expressed by equations (2.4) and (2.5). For stress-strain relationships of concrete, they used two types, i.e. the non-softened concrete stress-strain curve by CEB-FIP (1978) and the softened concrete stress-strain curve by Vecchio and Collins (1981), in order to evaluate the importance of taking

into account the softened behaviour of concrete. These stress-strain curves are displayed in Figure 2.17. In addition, for stress-strain curve of steel bars, it was assumed to be elastic perfectly plastic.

Equilibrium conditions:

$$\tau = V/(bd_w) \quad (2.1)$$

$$(A_{sl}f_l)/(bd_w) = \tau \cot \alpha \quad (2.2)$$

$$\tau = \sigma_d \sin \alpha \cos \alpha \quad (2.3)$$

Compatibility conditions:

$$0.5\gamma = (\epsilon_l + \epsilon_d) \cot \alpha \quad (2.4)$$

$$0.5\gamma = (\epsilon_t + \epsilon_d) \tan \alpha \quad (2.5)$$

where:

τ = average shear stress

V = applied shear force in wall

b = width of web cross section

d_w = effective depth of wall

A_{sl} = longitudinal reinforcement area

f_l = stress in the longitudinal reinforcement

α = angle of diagonal concrete struts to longitudinal axis

σ_d = stress on the cross section of the diagonal concrete struts

γ = shear distortion in wall

ϵ_d = compressive strain in the diagonal concrete struts

ϵ_l = tensile strain in the longitudinal bars

ϵ_t = tensile strain in the transverse bars

Figure 2.16 Equilibrium of shear wall (Hsu and Mo 1985).

Figure 2.17 (a) CEB-FIP (1978) stress-strain curve of non-softened concrete;
(b) Vecchio and Collins (1981) stress-strain curve of softened concrete; as reported by Hsu and Mo (1985).

The term ϵ_0 refers to strain of concrete at maximum compressive stress (f'_c) which is usually taken as 0.002 while the term ϵ_p refers to strain of softened concrete at maximum softened compressive stress (f_p). The

softening coefficient, λ , can be expressed as simple as $[\sec \alpha]$ (Hsu and Mo 1985). Furthermore, in case of low rise shear walls, ϵ_t can be taken as zero following the assumption that rigid foundation restrains the shear wall from expanding in transverse direction.

The nominal shear strength of wall (V_n) can be calculated by solving the equations from equilibrium, compatibility, and constitutive laws of concrete. The procedure is as follows (Hsu and Mo 1985):

- a) Select initial value of ϵ_d and then assume α .
- b) Calculate λ , ϵ_l , σ_d .
- c) Calculate α using equilibrium equations (2.2 and 2.3) and values obtained from step b and check whether α is close enough to the assumed value previously.
- d) If it is not sufficiently close, then repeat steps a, b, and c.
- e) Calculate τ and γ . This will provide one set of values.
- f) Select next value of ϵ_d and then repeat steps a to e. This will provide another set of values.
- g) Repeat until ϵ_d reaches maximum value which is 0.0030 or 0.0035. Then shear stress versus shear distortion curve of the wall can be plotted. Hence, from this curve, the nominal shear strength of wall, V_n , can be determined.

Hsu and Mo (1985) compared the walls shear strength predictions from softened truss model with experimental results of low rise shear walls (Galletly 1952; Benjamin and Williams 1957; Barda et al. 1977) and the comparison shows good agreement between experimental results and the predicted strengths from softened truss model. Furthermore, they also reported that the results from non-softened stress-strain curve of concrete were very poor and thus emphasized the importance of taking into account the softened stress-strain curve of concrete.

Nonetheless, there were limitations set for the softened truss model regarding its applicability (Hsu and Mo 1985). The model was valid for low rise shear walls with boundary elements having height to length ratio less than or equal to one. Furthermore, at that time, the model was

validated only with walls having longitudinal reinforcement ratio ranging from 0.25 to 2.95 per cent and concrete strength varied from 16.3 to 35.9 MPa.

In the subsequent paper, Mau and Hsu (1986) simplified the iterative procedure in the preceding work for design purposes. The work aimed for direct solution without iteration to predict the shear strength of low rise walls. The details of the simplification as well as design examples can be found elsewhere (Mau and Hsu 1986). Furthermore, in the following year, Mau and Hsu (1987) extended the applicability of the softened truss model to predict the shear strength of RC framed wall panels with vertical loads. The modified softened truss model was verified with experimental results (Yamada et al. 1974; Tomii and Esaki 1984) and the comparison shows reasonably good agreement between the actual wall strengths and predicted values from the analytical model.

Further development of the compression field theory (CFT) was done by Vecchio and Collins (1986) to take into account the tensile stresses in cracked concrete. The modified theory was then named “modified compression field theory” (MCFT). In original CFT, since the tensile stresses in cracked concrete are ignored, hence the shear strength of elements without transverse reinforcement is predicted to be zero. On the other hand, the MCFT gives significant contribution of concrete (V_c) to the shear strength by taking into account tensile stresses in cracked concrete. Thus, it is able to predict the shear strength of elements without transverse reinforcement.

Gupta and Rangan (1998) utilized the MCFT with some modifications to calculate the shear strength of NSC and HSC walls. In their calculation, following assumptions were made:

- a) The lateral force is mostly carried by the central panel of wall.
- b) Effect of bending on the shear behaviour of the panel is neglected.
- c) The distribution of stresses in the central panel is assumed to be uniform. Furthermore, the stresses are assumed to act on the effective

shear area which is bounded by effective depth of wall (d_w) and wall thickness (t_w).

Typical element in cracked central panel of a wall is shown in Figure 2.18 with l-t axes refer to longitudinal and transverse axes of the wall panel. The equilibrium and compatibility conditions used are almost the same as the ones used in softened truss model (Hsu and Mo 1985). However, here, they are written in more general forms to account for tensile stress and strain in concrete (Hsu 1993). These equations are displayed on (2.6) to (2.11).

Equilibrium conditions:

$$\sigma_l = \sigma_d \cos^2 \alpha + \sigma_r \sin^2 \alpha + \rho_l f_l \quad (2.6)$$

$$\sigma_t = \sigma_d \sin^2 \alpha + \sigma_r \cos^2 \alpha + \rho_t f_t \quad (2.7)$$

$$\tau_{lt} = (\sigma_r - \sigma_d) \sin \alpha \cos \alpha \quad (2.8)$$

Compatibility conditions:

$$\varepsilon_l = \varepsilon_d \cos^2 \alpha + \varepsilon_r \sin^2 \alpha \quad (2.9)$$

$$\varepsilon_t = \varepsilon_d \sin^2 \alpha + \varepsilon_r \cos^2 \alpha \quad (2.10)$$

$$\gamma_{lt} = 2 (\varepsilon_r - \varepsilon_d) \sin \alpha \cos \alpha \quad (2.11)$$

where:

σ_l = normal stress in longitudinal axis, positive for tension

σ_t = normal stress in transverse axis, positive for tension

σ_d = principal stress in d axis, positive for tension

σ_r = principal stress in r axis, positive for tension

τ_{lt} = average shear stress in l-t coordinate system and is due to shear force acting on the wall

ρ_l = average longitudinal reinforcement ratio

ρ_t = average transverse reinforcement ratio

f_l = average stress in the longitudinal reinforcement

f_t = average stress in the transverse reinforcement

α = angle of diagonal concrete strut (d axis) to longitudinal axis

ε_l = average strain in longitudinal axis, positive for tension

ε_t = average strain in transverse axis, positive for tension

- ε_d = average principal strain in d axis, positive for tension
 ε_r = average principal strain in r axis, positive for tension
 γ_{lt} = average shear strain in the element in l-t coordinate system

Figure 2.18 Typical element in cracked central panel of a wall (Gupta and Rangan 1998).

The stress-strain relationship for concrete was the one proposed by Collins et al. (1993) for concrete strength up to 100 MPa. However, this curve is for un-softened concrete. Hence, to take into account the softening behaviour of concrete, softening factor (ζ) was taken from the one proposed by Vecchio (1992). The equations for stress-strain curve of concrete can be found from Gupta and Rangan (1998). In addition, the stress-strain relationship for steel was assumed to be elastic perfectly plastic.

Another modification done by Gupta and Rangan (1998) and probably the most significant one was regarding the average strut angle, α . In their calculation, the strut angle (α) was expressed as:

$$\tan \alpha = d_w / H_w \quad (2.12)$$

where d_w is the effective depth of wall (taken as the distance between centre to centre of boundary elements or 0.8 of wall length, L_w , in case of absence of boundary elements) and H_w is the wall height. However, there were certain limits set for the values of α . The angle α should not be taken

larger than calculated value when the transverse strain $\varepsilon_t = 0$ and should not be taken smaller than calculated value when the transverse stress $\sigma_t = 0$. These limits became the upper bound and lower bound values for the strut angle α proposed by Gupta and Rangan (1998). Equation (2.12) provides additional condition required to solve the equilibrium, compatibility, and stress-strain relationships for concrete and steel. Moreover, one of the reasons for introducing this equation was that measured values of ε_t were far from zero which was opposite with the assumption of Hsu and Mo (1985). In their softened truss model, Hsu and Mo (1985) assumed that ε_t can be taken as zero for low rise walls with height to length (H_w/L_w) ratio less than or equal to one whereas Gupta and Rangan (1998) found that measured values of ε_t were far from zero in their specimens with H_w/L_w ratio of one. Nevertheless, Gupta and Rangan (1998) reported that their proposed method yielded good correlation with experimental results of walls failing in shear with wide range of H_w/L_w ratios, including those with H_w/L_w ratio more than one.

Another different procedure to take into account tensile stresses in cracked concrete was proposed by Hsu and his colleagues (Belarbi and Hsu 1994; Belarbi and Hsu 1995; Pang and Hsu 1995). Furthermore, the constitutive laws of concrete and steel bars have been modified such that they relate the average stresses to average strains in order to match with the assumption of continuous material as in equilibrium and compatibility conditions. The modified softened truss model was then called rotating angle softened truss model (RA-STM). The RA-STM utilizes the same equilibrium and compatibility conditions as displayed in equations (2.6) to (2.11). Moreover, the details of constitutive laws for concrete and steel bars used in RA-STM can be found elsewhere (Hsu 1996).

Mansour et al. (2004) used RA-STM with modification in the constitutive laws of materials to do an analytical study of nonlinear response of RC low rise shear walls. They collected data of thirty three RC low rise shear walls failing in web crushing mode available in literature (Galletly 1952; Benjamin and Williams 1957; Hirose 1957;

Barda et al. 1977; Cardenas et al. 1980; Ghoneim 1996). The specimens are mostly RC low rise shear walls with boundary elements having height to length ratio less than or equal to one. Moreover, concrete strength is varied from 16 to 43 MPa and the range of longitudinal reinforcement ratio is from 0.25 to 2.90 per cent. These characteristics of wall specimens would then become the range of applicability of the method proposed in their study. It is worth noting that the range of applicability of the proposed method here is almost the same as that of Hsu and Mo (1985). This is because the basic theory used in these two studies is same (softened truss model).

As in the case of RA-STM, here, the angle of principal compressive stress was assumed to coincide with the angle of principal compressive strain. Moreover, stresses and strains were assumed to be uniform throughout the wall panel so that concept of continuous materials could be applied. Hence, the equilibrium and compatibility conditions could be expressed in general forms as displayed on equations (2.6) to (2.11). However, since the proposed method was intended to predict the cyclic response of RC low rise shear walls, thus the constitutive laws of materials used were different with the original RA-STM. Here, cyclic constitutive laws of concrete and steel bars derived from full scale tests of reinforced concrete panels (Mansour et al. 2001) were used. They consist of positive (tension) and negative (compression) backbone curves as well as unloading and reloading curves. The details about these constitutive laws of materials can be found elsewhere (Mansour et al. 2001).

In this method, transverse strain (ϵ_t) was assumed to be zero as in the case of Hsu and Mo (1985) since large foundation was assumed to restrain the wall from expanding in transverse direction. In addition, it was also assumed that the imposed axial loads were resisted by the boundary elements only. Moreover, since the specimens investigated here were RC low rise shear walls having height to length ratio less than or equal to one, it was therefore assumed that the bending effect could be neglected. In this case, the wall web panel was subjected to pure shear

and thus the concept of RA-STM could be applied. The details of the analysis procedure can be found elsewhere (Mansour et al. 2004).

The analysis results (Mansour et al. 2004) show that the proposed method can predict the cyclic response of RC low rise shear walls with reasonable accuracy. For specimens subjected to monotonic loading, the proposed method predicts more accurately as compared to specimens subjected to cyclic loading. In case of cyclically loaded specimens, the proposed method slightly overestimates the maximum wall strengths. This is because in reality, cyclically loaded specimens suffer more damage as compared to monotonically loaded specimens while this phenomenon (cyclic degradation) is not accounted in the constitutive laws of materials used here. Moreover, due to the same reason, the proposed method underestimates the deflection and overestimates the energy dissipation capacity of cyclically loaded specimens.

Recent study by Kassem and Elsheikh (2010) aimed to address the issue of strut cracking angle (α) that had been previously dealt by Gupta and Rangan (1998). As previously mentioned, Gupta and Rangan (1998) proposed that the cracking angle could be taken as a function of geometric properties and they gave the lower as well as upper bound limits for the cracking angle values. However, earlier assessment of this fixed angle method by Yu and Hwang (2005) reported that the method inherited certain drawbacks, i.e. unreasonably high clamping stresses in the web and incorrect predictions of failure mode. Thus, Kassem and Elsheikh (2010) proposed a new method to estimate the strut cracking angle based on regression analysis of maximum shear capacity from available experimental results of walls subjected to monotonic and cyclic loadings. Moreover, they also took into account parameters affecting wall strengths such as geometric properties, reinforcement ratios, axial load ratio, and concrete strength.

In their analysis, equilibrium and compatibility conditions were expressed in general forms as displayed in equations (2.6) to (2.11). Furthermore, the compression stress-strain curve for softened concrete

used here was the one proposed by Zhang and Hsu (1998) with the softening factor (ζ) as defined by Yu and Hwang (2005) while the tension stress-strain curve for concrete used here was the one proposed by Gupta and Rangan (1998). In addition, the stress-strain curve for steel bars was assumed to be elastic perfectly plastic.

The regression analysis (Kassem and Elsheikh 2010) was based on available experimental results of 100 wall specimens subjected to monotonic and cyclic loadings. The analysis resulted into two equations which provide the values of cracking angle (α) for two different conditions, i.e. walls subjected to lateral and axial loading and walls subjected to lateral loading only. These equations are displayed on (2.13) and (2.14) as follows:

For walls subjected to lateral and axial loading:

$$\alpha = 57.16x_1^2 - 117.60x_1 + 87.10 \quad (2.13)$$

For walls subjected to lateral loading only:

$$\alpha = -7.34x_2^2 + 65.74x_2 - 87.67 \quad (2.14)$$

where:

$$x_1 = (H_w/d_w)(\sqrt{f'_c} P/A_g)^{0.1} \quad (2.15)$$

$$x_2 = \sqrt{f'_c} + (\rho_l/f_{yl})/(\rho_t/f_{yt})^{0.1} \quad (2.16)$$

Details of analysis procedure can be found elsewhere (Kassem and Elsheikh 2010). The analysis results show that the proposed method can predict load-deformation of walls subjected to monotonic and cyclic loadings and tested under lateral and axial loadings or pure lateral loading. Moreover, the cracking angles (α) predicted by equations (2.13) or (2.14) are shown to be reasonably accurate when compared with experimental results. In addition, shear capacities of walls predicted by the proposed method are more accurate than those predicted using fixed angle method proposed by Gupta and Rangan (1998) and design formula of ACI 318 (2005).

2.5.2 Softened Strut and Tie Model

The softened strut and tie model was originally proposed by Hwang and Lee (1999; 2000) for predicting the shear strength of beam-column joints. Later, Hwang et al. (2001) applied the model for predicting the shear strength of squat walls. They assumed that internal stress distribution was highly disturbed in squat walls and thus it was inappropriate to assume uniform stress distribution as in the case of softened truss model. In addition, they agreed that strut and tie model was a better approach to model the internal stress flow within squat walls.

The softened strut and tie model is based on truss analogy and it was derived to satisfy three basic principles of mechanic of materials, i.e. equilibrium, compatibility, and softened constitutive laws of cracked concrete. As in the case of softened truss model, the softening behaviour of cracked concrete is also highlighted in the softened strut and tie model. In this model, the applied external forces were assumed to be resisted by combination of concrete compression struts and steel tension ties. Three load paths in squat walls as modelled by Hwang et al. (2001) are displayed in Figure 2.19. They are diagonal, horizontal, and vertical mechanisms. It was assumed that the external horizontal concentrated load, V_{wh} , was divided among three mechanisms with certain ratios, i.e. R_d (diagonal ratio), R_h (horizontal ratio), and R_v (vertical ratio), in proportion to their relative stiffness (Hwang and Lee 1999; 2000). These stiffness ratios were defined according to the ones proposed by Jennewein and Schäfer (1992) and Schäfer (1996). Further details of the formulation of these ratios can be found elsewhere (Hwang et al. 2001).

Forces equilibrium of the softened strut and tie model is displayed on Figure 2.20 and is expressed in equations (2.17) and (2.18) while the compatibility condition is expressed in equation (2.19). It was assumed that the failure would be governed by maximum compressive stress acting on the nodal zone ($\sigma_{d,max}$) which was the summation of compressive stresses from diagonal, horizontal, and vertical struts as displayed in equation (2.20). Moreover, the constitutive law of the

softened concrete used here was the one proposed by Zhang and Hsu (1998) while the constitutive law of steel bars was assumed to be elastic perfectly plastic.

Figure 2.19 Load paths in softened strut and tie model for squat walls (Hwang et al. 2001).

Figure 2.20 Forces equilibrium in softened strut and tie model for squat walls (Hwang et al. 2001).

Equilibrium conditions:

$$V_{vv} = -D \sin \theta + F_h \tan \theta + F_v \quad (2.17)$$

$$V_{wh} = -D \cos \theta + F_h + F_v \cot \theta \quad (2.18)$$

Compatibility condition:

$$\varepsilon_r + \varepsilon_d = \varepsilon_h + \varepsilon_v \quad (2.19)$$

Maximum compressive stress acting on nodal zone:

$$\sigma_{d,max} = \frac{1}{A_{str}} \left\{ D - F_h \frac{\cos[\theta - \tan^{-1}(\frac{H'}{2\ell})]}{\cos[\tan^{-1}(\frac{H'}{2\ell})]} - F_v \frac{\cos[\tan^{-1}(\frac{2H'}{\ell}) - \theta]}{\sin[\tan^{-1}(\frac{2H'}{\ell})]} \right\} \quad (2.20)$$

where:

- R_d = wall shear ratio resisted by diagonal mechanism
- R_h = wall shear ratio resisted by horizontal mechanism
- R_v = wall shear ratio resisted by vertical mechanism
- V_{wv} = vertical wall shear force
- V_{wh} = horizontal wall shear force
- D = compression force in the diagonal strut, positive for tension
- F_h = tension force in the horizontal tie, positive for tension
- F_v = tension force in the vertical tie, positive for tension
- ε_r = average principal strain in r axis, positive for tension
- ε_d = average principal strain in d axis, positive for tension
- ε_h = average normal strain in horizontal axis, positive for tension
- ε_v = average normal strain in vertical axis, positive for tension
- $\sigma_{d,max}$ = maximum compressive stress acting on nodal zone
- θ = angle of inclination of diagonal compression strut which is defined here as $\tan^{-1}(H'/\ell)$
- H' = distance from point of application of V_{wh} to wall base
- ℓ = internal level arm of the couple forces at wall base
- a_s = depth of the diagonal strut
- b_s = width of the diagonal strut which can be taken as the wall web thickness (t_w)
- A_{str} = effective area of the diagonal strut which is defined as strut depth multiply by strut width ($a_s \times b_s$)

In the softened strut and tie model (Hwang et al. 2001), the strut depth (a_s) could be taken as the depth of compression zone at the wall base which together with the internal lever arm (ℓ) were determined from analysis of fully cracked transformed section with straight line theory. Furthermore, the effect of axial loading on wall shear strength was considered as additional amount of shear (longitudinal) reinforcement for

the case of axial loading applied at wall web as suggested by Mau and Hsu (1987) or was modelled as to increase the strut depth for the case of axial loading applied at the boundary elements.

The analysis procedure of the softened strut and tie model (Hwang et al. 2001) is explained as follows. Firstly, certain value of horizontal wall shear force (V_{wh}) is selected as initial value. Then, with equilibrium equations, the forces in struts and ties (D , F_h , and F_v) can be calculated as well as the maximum compressive stress acting on nodal zone ($\sigma_{d,max}$). Secondly, the initial value of softening factor (ζ) can be calculated as $\zeta = (-\sigma_{d,max})/f'_c$. Then, with constitutive laws of materials, the strains of struts and ties (ϵ_d , ϵ_h , and ϵ_v) can be calculated. Lastly, ϵ_r can be determined using compatibility equation and the new value of ζ is calculated and compared with its initial value. If the new calculated value of ζ is close enough with initial value, then V_{wh} selected is the shear strength of the wall. Otherwise, the analysis has to be repeated with different values of V_{wh} .

The analysis results (Hwang et al. 2001) show that the softened strut and tie model is able to predict the shear strength of RC squat walls failing in diagonal compression failures with reasonable accuracy. Furthermore, the model predicts more accurately as compared to ACI code (ACI Committee 318 1995). In addition, since the model consists of three load paths, it is able to incorporate explicitly the contribution from shear reinforcement to the shear strength of wall. Thus, the shear strength predicted is higher than that of contribution from diagonal strut alone. Nonetheless, the model underestimates the shear strength of RC squat walls with boundary elements while it predicts more accurately for rectangular ones.

In the subsequent paper, Hwang and Lee (2002) simplified the iterative procedure for design purposes. They aimed to reduce the number of variables involved and yet still able to sufficiently define the problems. They followed the frame of design equation that was previously proposed by Zhang and Jirsa (1982). However, instead of using empirically

determined strut and tie index (K) and softening factor (ζ), Hwang and Lee (2002) derived these variables theoretically based on shear resisting mechanisms of the softened strut and tie model and greatly simplified them. Here, the strut and tie index (K) was implemented to take into account the contribution from shear reinforcements whereas the softening factor (ζ) was implemented to take into account the softening behaviour of cracked concrete. Details of the formulations can be found elsewhere (Hwang and Lee 2002). They reported that the simplified approach was able to achieve similar degree of accuracy as the general approach.

Further study about application of softened strut and tie model for predicting the strength of double curvature RC squat walls was done by Bali and Hwang (2007). In this case, the basic theory and formulations were almost the same as those of single curvature RC squat walls (Hwang et al. 2001). However, definitions of some variables were adjusted to deal with the double curvature walls. Furthermore, in this study, not only for predicting the ultimate strength of the walls, they also proposed a method to predict the ultimate deflection by means of superposition of deflections caused by flexure, shear, and slip. The modifications of variables as well as the method to predict ultimate deflection of the double curvature walls are explained elsewhere (Bali and Hwang 2007).

Experimental verification was done with experimental results on double curvature RC squat walls (Lopes 1991; Hidalgo et al. 2002). The comparison between actual strengths and predicted strengths shows that the softened strut and tie model can predict reasonably well the shear strength of double curvature RC squat walls. However, for the ultimate deflection, it seems that the proposed method resulted in large variation of normalized actual deflections over calculated deflections.

2.5.3 General Comments

Based on the review of two major analytical models for predicting shear strength of RC walls, some general comments are summarized as follows.

Both softened truss model and softened strut and tie model provide rational approaches to be used for calculating the shear strength of RC walls rather than empirical equations given in most building codes. Furthermore, their capability to incorporate three basic principles of mechanics of materials (equilibrium, compatibility, and softened constitutive laws of cracked concrete) into their formulation is regarded as a main advantage over empirical building code formulas. Moreover, the researchers have shown that their models are capable of predicting the shear strength of RC walls by comparing the analysis results with experimental results from available literature. In addition, they also have simplified their models so that the models can be used practically by engineers.

However, despite their successful implementation in predicting the shear strength of RC walls, there is a fundamental difference between these two models. Softened truss model assumes that stresses and strains are distributed uniformly within the wall panel. On the other hand, softened strut and tie model assumes that stresses and strains are concentrated mainly in the compression struts and tension ties. As a result, these assumptions lead to differences in their mathematical formulations and their approaches in predicting the shear strength of RC walls.

The limitation of strut and tie model is that as it generally satisfies equilibrium conditions only, thus it allows engineers to choose different load paths for a particular problem. As a result, there may not be a unique solution for any particular case (ASCE-ACI Committee 445 1998). In fact, shear resisting mechanism that consists of three load paths in the softened strut and tie model as suggested by Hwang et al. (2001) can be considered as one of many possible shear resisting mechanisms of RC squat walls. Thus, there may exist other possible shear resisting mechanisms which may provide better solution, for example, a strut and tie model which can incorporate explicitly axial load applied at wall to the overall strut and tie model rather than to account the axial load as

additional shear (longitudinal) reinforcement as suggested by Hwang et al. (2001).

Another limitation of softened strut and tie model is its inability to predict load-deformation relationship of RC squat walls (Kassem and Elsheikh 2010). Although it satisfies one compatibility condition, the model does not give the values of wall deformation directly. Bali and Hwang (2007) proposed a method to determine the ultimate deflection of double curvature RC squat walls. Yet, the method results in large coefficient of variation of normalized actual deflections over calculated deflections.

On the other hand, softened truss model provides a unified concept for shear and torsion and it is considered the most promising method to treat shear and torsion (Mo and Rothert 1997). The model assumes that stresses and strains are uniformly distributed within the wall panel and thus the principle of continuum mechanics can be applied. Furthermore, the model is able to predict the complete load-deformation relationship of RC walls subjected to either monotonic or cyclic lateral loading. However, for the case of RC walls subjected to cyclic loading, the effect of concrete strength degradation due to cyclic loading needs to be incorporated to the softened constitutive laws of cracked concrete in order to get better prediction of wall strength, deformation, and energy dissipation capacity (Mansour et al. 2004).

Despite its superiority, softened truss model also has limitations. From studies by researchers (Hsu and Mo 1985; Mansour et al. 2004), it is concluded that the softened truss model is capable only of predicting the shear strength of low rise RC walls having height to length ratio less than or equal to one. In this case, the flexural deformation is so small and thus can be neglected. As a result, the wall panel is assumed to be subjected to state of pure shear. For RC walls with height to length ratio more than one, the flexural deformation becomes more pronounced and hence the model is unsuitable (Hsu and Mo 1985). Moreover, in the softened truss model, there is also uncertainty in determining the strut cracking angle

which significantly affects the shear strength predictions. Fixed angle solution proposed by Gupta and Rangan (1998) is able to give adequate estimation of the strut cracking angle. Nevertheless, as reviewed by Yu and Hwang (2005), the assumption of fixed angle method may be inconsistent with the assumption of uniform stress distribution in case of very low RC squat walls with high vertical loads. Kassem and Elsheikh (2010) also proposed a method to determine the strut cracking angle. Yet, the method is based on statistical analysis rather than a rational theory. In addition, most of the time, softened truss model requires iterations in order to obtain numerical solutions which may not be practical to be used directly by engineers.

In current study, an analytical model is developed based on softened truss model with some adjustments to eliminate the need for iterations to obtain numerical solutions. The model is intended to serve as an accurate method that can be used directly by engineers to predict the shear strength of RC walls. Furthermore, the results of this experimental study are used as the basis for certain assumptions in the proposed truss model as well as to improve its accuracy and applicability. In addition, the proposed truss model is verified with a wide range of experimental results of RC walls failing in shear including NSC and HSC walls which is the main concern of this study.

In addition to the proposed truss model, an analytical model utilizing strut and tie concept is also developed as an alternate method for predicting the shear strength of RC walls. In this model, different shear resisting mechanism as proposed by Hwang et al. (2001) is used. Moreover, the effect of axial loading, height to length ratio of walls, and the reinforced boundary elements to the shear strength of RC walls are also taken into account in the model. The proposed strut and tie model is developed with the aid of nonlinear finite element analysis in order to obtain the depth of compression zone at the bottom of wall which is the basis for determining the overall strut width. The shear strength predictions of the model are also verified with a wide range of

experimental results of RC walls failing in shear to ensure its accuracy and applicability.

2.6 Building Code Provisions

In this section, building code provisions to calculate nominal shear strength of RC walls are presented. In this study, building codes considered are ACI 318 code (2014) and Eurocode 8 (2004). The code provisions for calculating nominal shear strength of RC walls are briefly explained as follows.

2.6.1 ACI 318 (Chapter 18)

According to ACI 318 chapter 18 (ACI Committee 318 2014), the nominal shear strength of special structural walls can be calculated as follow:

$$V_n = A_{cv}(\alpha_c \lambda' \sqrt{f'_c} + \rho_t f_{yt}) \quad (2.21)$$

where:

V_n = nominal wall shear strength (N)

A_{cv} = gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered (mm²)

α_c = coefficient defining the relative contribution of concrete strength to nominal wall shear strength, which may be taken as 0.25 for $H_w/L_w \leq 1.5$, 0.17 for $H_w/L_w \geq 2.0$, and varies linearly between 0.25 and 0.17 for H_w/L_w between 1.5 and 2.0; where H_w/L_w is the height to length ratio of the wall

λ' = modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normal weight concrete of the same compressive strength

f'_c = specified compressive strength of concrete (MPa)

ρ_t = ratio of area of distributed transverse (horizontal) reinforcement to gross concrete area perpendicular to that reinforcement

f_{yt} = specified yield strength of transverse (horizontal) reinforcement (MPa)

Furthermore, ACI 318 also limits the maximum shear stress that can be resisted by wall to $0.83\sqrt{f'_c}$ to account for web crushing failure mode.

2.6.2 Eurocode 8

In this part, the nominal shear strength of RC walls is taken as the minimum shear strength between diagonal tension failure of the web and diagonal compression failure of the web according to Eurocode 8 (Comite Europeen de Normalisation 2004). The formulas are given as follows:

- For diagonal compression failure of the web due to shear:

$$V_n = \alpha_{cw} b z v_1 f'_c (\cot \theta_s + \tan \theta_s)^{-1} \quad (2.22)$$

where:

V_n = nominal wall shear strength (N), which for the critical region, it may be taken as 40% of the calculated value

α_{cw} = coefficient taking account of the state of the stress in the compression chord, which may be taken as 1.0 for non-prestressed structures; $[1 + P/(f'_c A_g)]$ for $0 < P/(f'_c A_g) \leq 0.25$; 1.25 for $0.25 < P/(f'_c A_g) \leq 0.5$; or $2.5 [1 - P/(f'_c A_g)]$ for $0.5 < P/(f'_c A_g) < 1.0$

P = applied axial force in wall, positive for compression (N)

f'_c = cylinder compressive strength of concrete (MPa)

A_g = gross cross section area (mm²)

b = width of web cross section (mm)

z = inner lever arm, for a member with constant depth, corresponding to the bending moment in the element under consideration, which may be taken equal to $0.8L_w$ (wall length) (mm)

v_1 = strength reduction factor for concrete cracked in shear, which is 0.6 $(1.0 - f'_c/250)$

$\cot \theta_s = \tan \theta_s = 1.0$

- For diagonal tension failure of the web due to shear:

If $M/(VL_w) \geq 2.0$:

$$V_n = z b \rho_h f_{yh} \cot \theta_s \quad (2.23)$$

If $M/(VL_w) < 2.0$:

$$V_n = b d [C_{Rd,c} k (100 \rho f'_c)^{1/3} + k_1 \sigma_{cp}] + 0.75 b \rho_h f_{yh} M/V \quad (2.24)$$

where:

V_n = nominal wall shear strength (N)

z = $0.8L_w$

L_w = wall length (mm)

b = width of web cross section (mm)

ρ_h = horizontal web reinforcement ratio

f_{yh} = yield strength of horizontal web reinforcement (MPa)

$\cot \theta_s = 1.0$

d = effective depth of cross section (mm)

$C_{Rd,c} = 0.18/\gamma_c$, which γ_c is taken as 1.0 for nominal strength without reduction factor for material

$k = 1 + \sqrt{(200/d)} \leq 2.0$

$\rho = A_s/(bd) \leq 0.02$ with A_s is the area of tensile reinforcement

f'_c = cylinder compressive strength of concrete (MPa)

$k_1 = 0.15$

$\sigma_{cp} = P/A_g < 0.2 f'_c$ (MPa)

P = applied axial force in wall, positive for compression (N)

A_g = gross cross section area (mm²)

M = applied bending moment in wall (Nmm)

V = applied shear force in wall (N)

2.6.3 General Comments

In this part, some general comments regarding building codes considered are briefly described based on recent analytical studies by the author (Chandra et al. 2011; Chandra and Teng 2012) while more detailed discussions are explained further in Chapter 5.

Based on those analytical studies, it was concluded that the flexure strength of RC walls could be predicted reasonably accurate using flexural theory for member subjected to axial load and bending moment.

However, for the shear strength of RC walls, building code formulas underestimated by significant margin, especially for HSC walls. It is generally understood that most building codes tend to give lower predictions of actual shear strength so that the design formulas are safe enough to be used for practical design.

The underestimation of building codes could be caused by a few inaccuracies in the shear strength formulas, but two factors were highlighted. One was the neglected contribution of vertical web reinforcement to wall shear strength. Another one was the limitation on the maximum shear strength values, which seemed to be quite conservative for HSC walls.

CHAPTER 3

ANALYTICAL STUDY ON SHEAR BEHAVIOUR OF RC WALLS

3.1 Overview

In this chapter, analytical study on shear behaviour of RC walls is presented. Data from past experiments of RC walls failing in shear are collected and studied. Normalized experimental wall shear strengths are plotted against several key parameters, such as height to length ratio, axial load ratio, web reinforcement ratio, concrete strength, etc., in order to see further the shear behaviour of RC structural walls as influenced by those parameters.

3.2 Data Collected from Past Experiments on RC Walls

Data from past experiments on RC walls failing in shear mode are collected and presented in terms of specimen ID, concrete compressive strength (f'_c), wall height to length ratio (H_w/L_w), axial load ratio ($P/[f'_c A_g]$; where P is the applied axial force in wall and A_g is the gross cross section area of wall), vertical and horizontal web reinforcement contributions ($\rho_v f_{yv}$ and $\rho_h f_{yh}$; where ρ_v and ρ_h are vertical and horizontal web reinforcement ratios of wall, f_{yv} and f_{yh} are the yield strengths of vertical and horizontal web reinforcement), experimental wall shear strength (V_{exp}), and drift ratio (%). There are a total of 77 wall specimens in the data collected from past experiments on RC walls failing in shear. Combined with data from this experimental study, there are a total of 84 wall specimens collected and they are presented in Table 3.1.

Table 3.1 Data of RC walls failing in shear mode collected from past experiments and the author's experimental study.

No.	Specimen ID [1]	f'_c (MPa) [2]	H_w/L_w [3]	$P/[f'_c A_g]$ [4]	$\rho_v f_{yv}$ (MPa) [5]	$\rho_h f_{yh}$ (MPa) [6]	V_{exp} (kN) [7]	Drift (%) [8]
(Barda et al. 1977)								
1	B1-1	29.0	0.46	0.00	2.72	2.48	1218	0.61
2	B2-1	16.3	0.46	0.00	2.76	2.50	978	0.69
3	B3-2	27.0	0.46	0.00	2.72	2.56	1108	0.56
4	B6-4	21.2	0.46	0.00	1.24	2.48	876	0.61
5	B7-5	25.7	0.21	0.00	2.65	2.51	1140	0.85
6	B8-5	23.4	0.96	0.00	2.64	2.48	886	0.56
(Cardenas et al. 1980)								
7	SW-7	43.0	1.00	0.00	3.44	1.12	519	0.45
8	SW-8	42.5	1.00	0.00	13.45	1.26	570	0.56
(Corley et al. 1981)								
9	B2	53.6	2.40	0.00	1.54	3.35	680	2.20
10	B5	45.3	2.40	0.00	1.46	3.16	762	2.20
11	B6	21.8	2.40	0.14	1.48	3.22	825	1.10
12	B7	49.4	2.40	0.08	1.42	3.08	980	2.80
13	B8	42.0	2.40	0.09	1.32	6.65	978	2.20
14	B9	44.1	2.40	0.09	1.34	2.91	977	2.20
15	B10	45.6	2.40	0.09	1.35	2.92	707	2.20
16	F1	38.5	2.40	0.00	1.58	3.73	836	1.10
17	F2	45.6	2.40	0.08	1.44	2.92	887	1.70
(Maeda 1986)								
18	MAE03	58.3	0.55	0.03	3.83	3.83	1460	0.62
19	MAE07	58.1	0.55	0.03	6.42	6.42	1676	0.59
(Okamoto 1990)								
20	W48M6	82.3	0.74	0.02	4.44	4.44	1516	0.60
21	W48M4	82.3	0.74	0.02	4.12	4.12	1479	1.01
22	W72M8	82.3	0.74	0.02	7.24	7.24	2066	1.01
23	W72M6	82.3	0.74	0.02	6.65	6.65	2015	1.02
24	W72M8	101.8	0.74	0.02	7.24	7.24	2128	1.01
25	W96M8	101.8	0.74	0.02	9.41	9.41	2483	1.02
(Mo and Chan 1996)								
26	HN4-1	32.2	0.58	0.00	2.20	2.46	205	0.36
27	HN4-2	32.2	0.58	0.00	2.20	2.46	247	1.38
28	HN4-3	32.1	0.58	0.00	2.20	2.46	202	0.44
29	HN6-1	29.5	0.58	0.00	3.22	3.61	255	1.04
30	HN6-2	29.5	0.58	0.00	3.22	3.61	204	0.26
31	HN6-3	31.0	0.58	0.00	3.22	3.61	205	0.56
32	HM4-1	37.5	0.58	0.00	2.20	2.46	223	0.44
33	HM4-2	37.5	0.58	0.00	2.20	2.46	231	0.36
34	HM4-3	39.9	0.58	0.00	2.20	2.46	250	2.10
35	LN4-1	18.0	0.58	0.00	1.76	2.46	193	0.52
36	LN4-2	18.0	0.58	0.00	1.76	2.46	217	1.66

Table 3.1 Data of RC walls failing in shear mode collected from past experiments and the author's experimental study (continued).

No.	Specimen ID [1]	f'_c (MPa) [2]	H_w/L_w [3]	$P/[f'_c A_g]$ [4]	$\rho_v f_{yv}$ (MPa) [5]	$\rho_h f_{yh}$ (MPa) [6]	V_{exp} (kN) [7]	Drift (%) [8]
37	LN4-3	29.7	0.58	0.00	1.76	2.46	203	1.12
38	LN6-1	30.7	0.58	0.00	2.58	3.61	246	0.54
39	LN6-2	30.2	0.58	0.00	2.58	3.61	200	0.24
40	LN6-3	30.2	0.58	0.00	2.58	3.61	210	0.26
41	LM6-1	39.3	0.58	0.00	2.58	3.61	219	0.42
42	LM6-2	37.0	0.58	0.00	2.58	3.61	205	0.28
43	LM6-3	34.5	0.58	0.00	2.58	3.61	210	0.00
44	LM4-2	66.0	0.58	0.00	1.76	2.46	250	0.94
45	LM4-3	66.0	0.58	0.00	1.76	2.46	227	0.36
(Gupta and Rangan 1998)								
46	S-1	79.3	1.00	0.00	5.45	2.89	428	1.61
47	S-2	65.1	1.00	0.07	5.45	2.89	720	1.11
48	S-3	69.0	1.00	0.13	5.45	2.89	851	0.56
49	S-4	75.2	1.00	0.00	8.00	2.89	600	1.21
50	S-5	73.1	1.00	0.06	8.00	2.89	790	0.78
51	S-6	70.5	1.00	0.13	8.00	2.89	970	0.74
52	S-7	71.2	1.00	0.06	5.45	5.45	800	0.94
(Kabeyasawa and Hiraishi 1998)								
53	W-08	103.3	1.18	0.09	5.75	5.75	1670	0.73
54	W-12	137.5	1.18	0.09	5.75	5.75	1719	0.78
55	No. 1	65.1	1.18	0.13	1.58	1.58	1101	0.71
56	No. 2	70.8	1.18	0.12	2.75	2.75	1255	0.70
57	No. 3	71.8	1.18	0.12	4.22	4.22	1379	0.76
58	No. 4	103.4	1.18	0.14	4.22	4.22	1697	0.72
59	No. 5	76.7	1.76	0.11	4.22	4.22	1159	1.00
60	No. 6	74.1	1.18	0.12	9.31	9.31	1412	0.72
61	No. 7	71.5	1.18	0.12	7.92	7.92	1499	0.74
62	No. 8	76.1	1.18	0.11	11.52	11.52	1639	0.76
(Farvashany et al. 2008)								
63	HSCW1	104.0	1.25	0.04	6.74	2.51	735	0.97
64	HSCW2	93.0	1.25	0.09	6.74	2.51	845	1.13
65	HSCW3	86.0	1.25	0.09	4.01	2.51	625	0.93
66	HSCW4	91.0	1.25	0.22	4.01	2.51	866	0.76
67	HSCW5	84.0	1.25	0.09	6.74	4.01	801	1.32
68	HSCW6	90.0	1.25	0.05	6.74	4.01	745	1.34
69	HSCW7	102.0	1.25	0.08	4.01	4.01	800	1.27
(Burgueno et al. 2014)								
70	M05C	46.0	2.25	0.08	6.54	8.14	803	1.77
71	M05M	38.9	2.25	0.09	6.54	8.14	855	1.77
72	M10C	56.4	2.25	0.06	7.00	8.71	751	1.68
73	M10M	84.0	2.25	0.04	7.00	8.71	900	5.12
74	M15C	102.0	2.25	0.03	7.07	8.80	819	3.10

Table 3.1 Data of RC walls failing in shear mode collected from past experiments and the author's experimental study (continued).

No.	Specimen ID [1]	f'_c (MPa) [2]	H_w/L_w [3]	$P/[f'_c A_g]$ [4]	$\rho_v f_{yv}$ (MPa) [5]	$\rho_h f_{yh}$ (MPa) [6]	V_{exp} (kN) [7]	Drift (%) [8]
75	M15M	111.0	2.25	0.03	7.03	8.75	934	5.24
76	M20C	131.0	2.25	0.03	6.44	10.69	815	3.01
77	M20M	115.0	2.25	0.03	6.44	10.69	923	7.44
Author								
78	J1	103.3	1.00	0.05	1.71	1.71	1210	0.79
79	J2	96.8	1.00	0.05	4.34	1.71	1271	0.68
80	J3	110.7	1.00	0.05	1.71	4.34	1459	0.76
81	J4	93.5	1.00	0.05	1.71	1.71	811	0.54
82	J5	103.3	2.00	0.05	1.71	1.71	596	0.70
83	J6	96.8	2.00	0.05	4.34	1.71	724	0.80
84	J7	110.7	2.00	0.05	1.71	4.34	895	1.17

3.3 Analytical Study on Shear Behaviour of RC Walls

The shear behaviour of RC walls is observed by plotting average shear stresses (experimental wall shear strengths divided by wall web area, A_w) normalized by the square root of concrete strength against key parameters such as height to length ratio (H_w/L_w), axial load ratio ($P/[f'_c A_g]$), concrete strength (f'_c), ratio of vertical reinforcement in boundary element (ρ_b), and vertical and horizontal web reinforcement contributions ($\rho_v f_{yv}$ and $\rho_h f_{yh}$). This is done for similar specimens that have the same properties except the parameters compared in order to see the relationship between normalized RC wall shear strengths and the parameters. The results are presented in Figures 3.1 to 3.5. Overall, the results of this analytical study are in agreement with lessons learned from past experiments which are presented in Chapter 2.

As can be seen in Figure 3.1, the normalized average shear stresses decrease as height to length ratio increases. This means that RC walls with higher height to length ratio have lower shear strengths as compared to RC walls with lower height to length ratio. Figure 3.2 shows that the normalized average shear stresses increase with increment in axial load ratio. This implies that RC walls subjected to higher axial stress have higher shear strengths as compared to RC walls subjected to lower axial stress.

Figure 3.1 Normalized average shear stresses plotted against height to length ratio.

Figure 3.2 Normalized average shear stresses plotted against axial load ratio.

Figure 3.3 Normalized average shear stresses plotted against concrete compressive strength.

Figure 3.4 Normalized average shear stresses plotted against ratio of vertical reinforcement in boundary element.

Figure 3.5 Normalized average shear stresses plotted against web reinforcement contributions.

For concrete compressive strength, Figure 3.3 shows that the normalized average shear stresses decrease with increment in concrete compressive strength. These results are in agreement with experimental results by Burgueno et al. (2014). In their experiment, it is concluded that increase in concrete compressive strength does not linearly increase web crushing strength. Instead, the relationship is nonlinear, with a decreasing limit as concrete strength increases. If the diagonal concrete struts remain undamaged, then web crushing strength increases in proportion to concrete strength. However, application of cyclic loading and presence of inelastic deformations will cause damage to diagonal concrete struts, and hence it limits web crushing strength independently of concrete strength. Thus, for RC walls subjected to cyclic loading as plotted in Figure 3.3, increase in concrete strength does not necessarily increase the shear strengths. Therefore, with similar shear strengths, RC walls with higher concrete compressive strength will have lower normalized average shear stresses as compared to RC walls with lower concrete compressive strength.

For ratio of vertical reinforcement in boundary element, it can be seen from Figure 3.4 that the normalized average shear stresses increase with increment of ratio of vertical reinforcement in boundary element. The increment of the normalized average shear stresses is more significant for RC walls with higher height to length ratio (specimens tested by Corley et al. (1981)). This is due to for such RC walls, the behaviour can be dominated by flexure deformation although they ultimately fail in shear. The addition of vertical reinforcement in boundary element will increase the flexural capacity of RC walls. Hence, if flexural yielding can be avoided, higher shear stresses can be achieved for web crushing (Corley et al. 1981).

For web reinforcement contributions, it can be seen from Figure 3.5 that the normalized average shear stresses increase with the increment of vertical and horizontal web reinforcement contributions. In the figure's legend, notes V and H refer to vertical and horizontal web reinforcement, respectively. From the figure, it can be concluded that both vertical and horizontal web reinforcement have contributions to the RC wall shear strengths. However, their effectiveness

depends on height to length ratio of the RC walls. It can be seen that for RC walls with height to length ratio less than 1.00, vertical web reinforcement is more effective than horizontal web reinforcement in contributing to the RC wall shear strengths while the opposite is true for RC walls with height to length ratio equal to or more than 1.00. This phenomenon is not taken into account in the building code formulas for calculating RC wall shear strengths. Code formulas only take into account the contribution from horizontal web reinforcement while neglecting the vertical web reinforcement contribution. Thus, it results in underestimation of RC wall shear strengths by ACI 318 code and Eurocode 8 (Chandra et al. 2011; Chandra and Teng 2012).

CHAPTER 4

EXPERIMENTAL STUDY

4.1 Overview

In this chapter, the experimental study conducted is explained in detail. Firstly, the objectives and scope of the experiment are introduced. Then, design of specimens including dimensions, reinforcement detailing, and their material properties are described in detail with supporting tables and figures. Moreover, some photographs of specimens' preparation such as steel cages preparation, formworks preparation, specimens casting, etc. are presented as well. Furthermore, the experimental setup is also given in terms of overall test setup, linear variable displacement transducers (LVDTs) setup, and strain gauges setup as well as the experiment loading history. Lastly, the experiment results are presented and discussed in detail.

4.2 Objectives and Scope

As presented in previous chapters, data of structural walls collected from literature were limited to walls with concrete strength up to 100 MPa. There are still very limited data for walls with concrete strength about 100 MPa and above. Since the use of HSC walls with concrete strength above 100 MPa is highly recommended for large underwater structures with expected service life of more than one or two hundred years, it is therefore necessary to investigate the behaviour of such HSC walls which may differ from NSC walls. Furthermore, experimental studies are the best representations of real structures rather than performing numerical studies with some computer models.

The results of recent analytical studies on structural walls by the author (Chandra et al. 2011; Chandra and Teng 2012) showed that the flexure behaviour (strength) of NSC and HSC walls could be well predicted using flexural theory whereas the shear behaviour (strength) could not be well predicted using empirical formulas from building codes. Moreover, shear behaviour is much more complicated and it has not been fully understood as compared to flexure behaviour. Hence, in this experimental study, the focus

was on the investigation of the shear behaviour of HSC walls and factors affecting it. Key factors investigated were height to length ratio, vertical and horizontal web reinforcement ratios, and specimen shape.

4.3 Design of Specimens

There were seven HSC wall specimens designed to achieve the experiment objectives. The characteristic cylinder concrete compressive strength of all specimens at 28 days was designed to be 100 MPa. Furthermore, all specimens were designed in such a way that the shear behaviour would govern the overall strength of the walls. Moreover, parameters such as height to length ratio, vertical and horizontal web reinforcement ratios, and specimen shape were varied to investigate the effect of each parameter on the shear behaviour of HSC walls. The axial load ratio ($P/[f'_c A_g]$) was kept constant at 0.05 for all specimens. This ratio was selected as it is within the possible range of axial load for walls in structures and most importantly it was within the capacity limit of loading equipment available as HSC walls required higher loads to achieve the same ratio as NSC walls. Since it was impossible to have full scale test of HSC walls due to the limitations of loading equipment capacity for axial loading as well as lateral loading, thus in this experiment the specimens were scaled to be one-third of typical RC walls in general buildings. The specimens' dimensions were adjusted to suit the capacity of loading equipment available in the Protective Engineering Laboratory (PE Lab) of Nanyang Technological University (NTU), Singapore. Detail of the specimens can be seen further in Table 4.1 and Figures 4.1 to 4.6.

Specimen J1 was designed as the control specimen. It had height to length ratio (H_w/L_w) of 1.0 which could be categorized as squat wall and vertical and horizontal web reinforcement ratios of 0.28 per cent which satisfied minimum requirements as specified by ACI 318 (2014) and Eurocode 8 (2004). For specimens J2 and J3, the parameters varied were vertical and horizontal web reinforcement ratios, respectively. In specimen J2, the vertical web reinforcement ratio (ρ_v) was increased to 0.75 per cent while in specimen J3, the same increase was done for the horizontal web reinforcement ratio (ρ_h).

Specimen J4 was designed with same height to length ratio and web reinforcement ratios as specimen J1, yet without the boundary elements to investigate the effect of boundary elements on the shear behaviour of HSC walls.

Specimen J5 was designed to investigate the effect of height to length ratio on the shear behaviour of HSC walls. It had height to length ratio (H_w/L_w) of 2.0 which could be categorized as slender wall. In a typical slender wall, the behaviour is usually dominated by flexure. However, in this experiment, the slender walls with height to length ratio of 2.0 were designed in such a way so that they would fail in shear mode. This was done by putting enough flexural reinforcement at the boundary elements so that the wall flexural strengths were higher than the wall shear strengths by sufficient margin to ensure shear failure. For this purpose, high strength rebar with nominal yield strength above 600 MPa was used as the vertical reinforcement in the boundary elements to keep the vertical reinforcement ratio within acceptable limit. Therefore, in order to be consistent, the same amount of vertical reinforcement in boundary elements was also provided for squat wall specimens. It is worth noting that for the case of slender wall, there was no specimen designed without boundary elements since the failure mode of such specimen would be governed by flexure behaviour and thus it was beyond the objectives and scope of this experimental study.

As in the case of specimens J2 and J3, specimens J6 and J7 were also designed to investigate the effect of web reinforcement ratios on the shear behaviour of HSC walls. Lessons learned from past experiments show that the contribution of vertical and horizontal web reinforcement to the shear strength of walls is varied with respect to height to length ratio. The vertical web reinforcement is more effective in contributing to the shear strength for squat walls whereas the horizontal web reinforcement is more effective for slender walls. Thus, in this experiment, the effect of web reinforcement ratios on the shear strength of HSC walls was investigated for different walls height to length ratio.

Table 4.1 Parameters variation in the wall specimens.

No.	Specimen ID	A_g (mm ²)	H_w/L_w	$P/[f'_c A_g]$	ρ_v	ρ_h	Boundary Elements	Variables
1	J1	196000	1.0	0.05	0.0028	0.0028	YES	Control specimen
2	J2	196000	1.0	0.05	0.0075	0.0028	YES	Vertical web reinforcement ratio
3	J3	196000	1.0	0.05	0.0028	0.0075	YES	Horizontal web reinforcement ratio
4	J4	111200	1.0	0.05	0.0028	0.0028	NO	Specimen shape
5	J5	196000	2.0	0.05	0.0028	0.0028	YES	Height to length ratio
6	J6	196000	2.0	0.05	0.0075	0.0028	YES	Height to length ratio and vertical web reinforcement ratio
7	J7	196000	2.0	0.05	0.0028	0.0075	YES	Height to length ratio and horizontal web reinforcement ratio

Figure 4.1 Dimensions of specimens J1-J3.

Figure 4.2 Reinforcement detailing for specimens J1-J3.

NOTES:

CONCRETE COVERS:
BEAMS = 40 MM
COLUMNS = 20 MM
WALLS = 20 MM

ALL DIMENSIONS ARE IN MM

Figure 4.3 Dimensions of specimen J4.

REINFORCEMENT DETAILING SPECIMEN J4

SECTION 1-1
SPECIMEN J4

SECTION 2-2
SPECIMEN J4

NOTES:

SPECIMEN J4:
A = 5x2D6@200
B = 6x2D6@200

ALL DIMENSIONS ARE IN MM

Figure 4.4 Reinforcement detailing for specimen J4.

Figure 4.5 Dimensions of specimens J5-J7.

Figure 4.6 Reinforcement detailing for specimens J5-J7.

4.4 Specimens Preparation

In this section, detail about specimens' preparation are explained including materials used as well as their properties, steel cages and formworks preparation, and specimens casting. Some photographs are presented as well to give better description about sequence of construction of the test specimens.

4.4.1 Material Properties

The properties of concrete and steel bars used in this experimental study are explained here in detail. This includes concrete mix design, testing of concrete cylinder samples, and testing of steel bars samples.

The concrete mix design used is presented in Table 4.2. The target concrete cylinder compressive strength at 28 days was 100 MPa. Thus, it needed low water to binder ratio and high cementitious materials content to achieve this objective. The selected water to binder ratio was 0.26 and cementitious materials content used was 600 kg/m³ concrete. The cementitious materials used consisted of 55% ordinary Portland cement, 35% ground granulated blast furnace slag (GGBS), and 10% uncondensed silica fume. The use of GGBS and silica fume in the concrete mix design was also for durability purpose since it is one of main concerns in this study to provide concrete for underwater (marine) structures with service life of more than one or two hundred years. Furthermore, super-plasticizer was also used in the mix design to ensure workability of the concrete with such low water to binder ratio and high cementitious materials content. Moreover, the maximum coarse aggregate size used was 10 mm to ensure good compaction of the concrete during casting.

Several concrete cylinder samples with diameter of 100 mm and height of 200 mm were cast from each mix to obtain concrete compressive strength as well as splitting tensile strength. Testing of these cylinders was done in several time intervals, most importantly at age of 28 days and at age of 91 days or above. It is worth noting that the wall specimens tested in this study were tested at age of more than 91 days. This was to

ensure that the concrete had already gained its optimum strength at the day of specimens testing. Complete results of concrete cylinders testing are presented on Table 4.3 while stress-strain curves of concrete cylinders are presented on Figure 4.7a.

Table 4.2 Concrete mix design for 1 m³ concrete.

Materials	Weight (kg)
Water	155
Coarse aggregate 10 mm	860
Fine aggregate (sand)	860
Ordinary Portland cement	330
Silica fume	60
Ground granulated blast furnace slag	210
Super plasticizer	9

Table 4.3 Concrete cylinders testing results.

Mix Number	Age (days)	f _c (MPa)	f _{ct} (MPa)
I (Specimens J1&J5)	14	93.1	N/A
	28	97.1	6.4
	91	103.3	N/A
II (Specimens J2&J6)	14	85.4	5.1
	28	94.7	6.2
	182	96.8	7.0
III (Specimens J3&J7)	14	103.1	6.7
	28	105.9	7.0
	119	110.7	7.1
IV (Specimen J4)	21	88.7	N/A
	28	93.4	N/A
	260	93.5	5.5

Figure 4.7a Stress-strain curves of concrete cylinders.

All steel bars used in this experimental study were deformed bars. Their complete properties can be seen in Table 4.4 such as nominal diameter, yield strength, yield strain, tensile strength, tensile strain, and modulus of elasticity. Furthermore, stress-strain curves of bar samples are presented on Figure 4.7b.

Table 4.4 Steel bars properties.

Bar ID	Nominal diameter (mm)	Yield strength (MPa)	Yield strain	Tensile strength (MPa)	Tensile strain	Modulus of elasticity (MPa)
D6	5.94	610	0.0031	637	0.0988	197488
D10	9.77	578	0.0033	652	0.0876	176339
D10'	9.88	617	0.0032	709	0.0794	192981
D13	12.82	592	0.0032	692	0.1000	186667
D16	15.72	630	0.0034	715	0.0952	185878
D20	19.81	591	0.0031	692	0.1186	189438

Figure 4.7b Stress-strain curves of bar samples.

4.4.2 Steel Cages and Formworks Preparation

Steel cages and formworks were prepared by a local contractor in Singapore. Some photographs of steel cages and formworks are displayed on Figures 4.8 and 4.9.

(a)

(b)

Figure 4.8 Steel cages of the specimens.

(a)

(b)

Figure 4.9 Formworks of the specimens.

4.4.3 Specimens Casting

Concrete used in this experimental study was supplied by a commercial ready mix plant in Singapore. The specimens were cast vertically to represent real construction practice for RC structural walls. First, the bottom beams were cast and left to dry for two hours. Then, the wall webs, boundary elements (if any), and top beams were cast simultaneously. Some photographs of specimens casting are presented in Figure 4.10. Formworks were removed within 3 days after casting and after that the specimens were covered using gunny sack and water cured for 14 days.

Figure 4.10 Specimens casting.

4.5 Experimental Setup

In this section, details about the experimental setup are explained including the loading test setup, LVDTs setup, strain gauges setup, and the loading history. Before performing the experiment, it is important to plan these setups properly since they will greatly affect the experimental results as well as the findings. Improper experimental setup may result in unreasonable or incomplete results and thus the experiment objectives may not be achieved.

4.5.1 Loading Test Setup

The wall specimens were loaded axially and laterally. The axial loading was applied using one hydraulic jack with 2000 kN capacity which was attached to a loading frame. The axial load from the hydraulic jack was transferred to the top beam of the wall specimens using a steel spreader beam to ensure uniform axial load distribution on the wall cross section area. Between the spreader beam and the hydraulic jack, there was a spherical bearing placed to allow rotational movement of the top beam during testing as well as a 2000 kN load cell to measure the axial load. Moreover, some steel rollers were placed below the spreader beam to accommodate horizontal movement of the wall specimens due to lateral loading.

The lateral loading was applied using two hydraulic actuators with 1000 kN capacity each. Thus, the total lateral loading that could be applied to the wall specimens was 2000 kN. At one end, the actuators were connected to laboratory reaction wall while at the other end, they were connected to a steel loading beam. The lateral load from the actuators was transferred to the top beam of the wall specimens using the steel loading beam and prestressing rods which were inserted to the top beam of the wall specimens. It was important to make sure that the two actuators were giving the same displacement values during the testing to avoid any eccentricity. In addition, to accommodate any rotation possible

because of wall deformation during testing, the steel loading beam was connected to the top beam of the wall specimens with hinge connection.

The wall specimens were connected to laboratory strong floor using eight prestressing rods which were prestressed for about 400 kN each to avoid any relative horizontal movement between wall bottom beam and the strong floor as well as to resist the overturning moment at the wall base. Furthermore, two restraining blocks were installed as well at each side of the bottom beam to help prevent horizontal movement. The schematic drawing of loading test setup can be seen in Figures 4.11 to 4.13 while some photographs are displayed on Figures 4.14 and 4.15.

4.5.2 Linear Variable Displacement Transducers (LVDTs) Setup

LVDTs were used to measure displacement response of the wall specimens during testing. LVDTs arrangement can be seen in Figures 4.16 to 4.18. There were four LVDTs (DT1-DT4) attached to the top beam of all specimens for measuring their lateral in-plane and out-of-plane displacements. The in-plane displacement was used to plot the force drift relationship of the specimens while the out-of-plane displacement was used to measure any possible movement in out-of-plane direction. Furthermore, there were four LVDTs (DT5-DT8) attached to the bottom beam for measuring any possible uplift and horizontal movements of the bottom beam. Moreover, for each specimen, two LVDTs (DT9 and DT10) were installed at the wall base to measure sliding shear deformation. For specimens J1-J4, there were two LVDTs attached to the wall web to measure shear deformation while for specimens J5-J7, four LVDTs were used for the same purpose. Flexural deformation of all specimens was measured by attaching LVDTs at both ends of the wall specimens. The gage length for LVDTs installed near the wall base was shorter than others since the flexural deformation was mostly concentrated at the wall base. There were a total of six LVDTs attached to both ends of specimens J1-J4 while for specimens J5-J7, there were a total of twelve LVDTs installed.

Figure 4.11 Elevation view of the loading test setup.

Figure 4.12 Plan view of the loading test setup.

Figure 4.13 Side view of the loading test setup.

Figure 4.14 Photo of test setup for specimens with height to length ratio of 1.0.

Figure 4.15 Photo of test setup for specimens with height to length ratio of 2.0.

NOTES:
ALL DIMENSIONS ARE IN MM

Figure 4.16 LVDTs setup for specimens J1-J3.

NOTES:
ALL DIMENSIONS ARE IN MM

Figure 4.17 LVDTs setup for specimen J4.

Figure 4.18 LVDTs setup for specimens J5-J7.

4.5.3 Strain Gauges Setup

Strain gauges were used to measure strain in reinforcement bars during the testing. In this experiment, steel strain gauges with 5 mm gage length were used. The strain gauges setup can be seen in Figures 4.19 to 4.21.

FRONT VIEW
STRAIN GAUGES
SETUP
SPECIMEN J1-J3

SIDE VIEW
STRAIN GAUGES
SETUP
SPECIMEN J1-J3

TOP VIEW
STRAIN GAUGES
SETUP
SPECIMEN J1-J3

Figure 4.19 Strain gauges setup for specimens J1-J3.

FRONT VIEW
STRAIN GAUGES
SETUP
SPECIMEN J4

SIDE VIEW
STRAIN GAUGES
SETUP
SPECIMEN J4

TOP VIEW
STRAIN GAUGES
SETUP
SPECIMEN J4

Figure 4.20 Strain gauges setup for specimen J4.

FRONT VIEW
STRAIN GAUGES
SETUP
SPECIMEN J5-J7

SIDE VIEW
STRAIN GAUGES
SETUP
SPECIMEN J5-J7

TOP VIEW
STRAIN GAUGES
SETUP
SPECIMEN J5-J7

Figure 4.21 Strain gauges setup for specimens J5-J7.

The strain gauge notations are explained as follows. The alphabets denote the grid location while the numbers denote the strain gauge numbering system. Grid location A-D is in the direction of wall length while grid location N-S is in the direction of wall thickness.

For all specimens, the strain gauges were distributed regularly in the vertical reinforcement as well as in the horizontal reinforcement. In vertical reinforcement, the strain gauges were installed in three locations, i.e. wall base, mid height of wall, and wall top. This was done to measure the strain distributions along the vertical reinforcement. For horizontal reinforcement, the strain gauges were also installed in three locations, i.e. left end, middle, and right end of the wall. This was also done to measure the strain distributions along the horizontal reinforcement.

For specimens J1-J3, the total number of strain gauges per wall specimen was sixty nine gauges. In the vertical reinforcement in boundary elements, the strain gauges were placed in grid A and D only. This was because grid B was very close to grid A and grid C was very close to grid D and thus the strain values were almost the same. In order to reduce the number of strain gauges, it was therefore decided to put the strain gauges on outer grids only. Furthermore, to investigate the difference of strains between vertical bars which were located near to the web and vertical bars which were far away from the web, thus the strain gauges were placed in several vertical bars in the boundary elements. Moreover, for stirrups in boundary elements, the strain gauges were installed at locations where horizontal reinforcement was present at the wall web.

For specimen J4, the total number of strain gauges in the specimen was also sixty nine gauges. However, the difference was in the location of strain gauges installed in vertical bars at boundary elements. Since there were no flanges in specimen J4, the same amount of vertical bars was distributed in the direction of wall length. Thus, the strains would be quite different between vertical bars in outer side (i.e. grid A and D) and vertical bars in inner side (i.e. grid B and C). In this case, the strain

gauges were installed at vertical bars in grid A, B, C, and D to measure the strain distributions.

For specimens J5-J7, the total number of strain gauges per wall specimen was eighty nine gauges. The increment in number of strain gauges needed was because of increment in number of horizontal reinforcement in the wall specimens. Furthermore, the strain gauges arrangement in boundary elements was the same as in the case of specimens J1-J3.

4.5.4 Loading History

Here, loading history refers to lateral cyclic loading history since the vertical axial loading was kept constant at ratio of 0.05 for all specimens during testing. The full axial loading had been applied first before lateral cyclic loading was applied.

To investigate the shear strength capacity of HSC walls, all specimens were tested until failure. The magnitudes of lateral cyclic loading were increased progressively using displacement control as shown in Figure 4.22. This loading history with increasing amplitudes is one of the most commonly used for quasi-static simulated earthquake loading tests (Derecho et al. 1980; Park 1988). Furthermore, it is worth noting that quasi-static loading usually gives conservative prediction of the actual strength (Park 1988). Nevertheless, Park (1988) also stated that the hysteretic behaviour might not differ between structures subjected to quasi-static loading and dynamic loading.

For the first and second cycle, the specimens were subjected to displacement equal to 0.05% and 0.10% drift ratio, respectively. After that, for subsequent cycles, the displacement was increased by 0.10% drift ratio until the displacement reached 1.00% of drift ratio. From this point forward, the displacement was increased by 0.20% progressively until it reached 2.00% of drift ratio or until failure.

Figure 4.22 Quasi-static cyclic loading history for all specimens.

4.6 Experiment Results

In this section, the experiment results are explained in detail including force-drift relationships, envelope curves, wall deformation components, and strain distributions. Moreover, some photographs explaining crack propagation stages during testing are also presented to give better understanding about how cracks were developed when the specimens were tested until failure. The results are presented here for each specimen one by one starting from specimen J1 to specimen J7 while the summary of the results as well as the discussions are presented in the next section.

4.6.1 Specimen J1

Concrete cylinder compressive strength for specimen J1 was taken from cylinder compressive strength tested at the age of 91 days although the specimen J1 itself was tested at a later date due to unavailable data of cylinder compressive strength at the day of specimen testing. However, since concrete usually reaches its optimum strength at the age of 91 days,

thus it can be assumed that the compressive strength development after 91 days is negligible. Based on the test at the age of 91 days, the concrete cylinder compressive strength for specimen J1 was 103.3 MPa.

For specimen J1, during testing there was a problem with the restraining block which was supposed to prevent movement of bottom beam in positive direction. At the sixth cycle, drift amplitude was supposed to be increased from +0.40% to +0.50%. However, starting from drift value of +0.40% onwards, any attempt to increase the drift value would cause excessive movement of bottom beam without any significant increment of lateral force. Thus, it was decided that the testing was continued monotonically in negative direction starting from sixth cycle onwards until the specimen failed. The crack propagation stages for specimen J1 are presented on Figure 4.23. For all specimens, please note that in photographs, positive loading direction is toward left direction of the wall while negative loading direction is toward right direction of the wall. For specimen J1, diagonal cracks started to occur in wall web as early as in the second cycle and the number of cracks increased in subsequent cycles. In boundary element flange, horizontal cracks started to occur in the third cycle and they were located mostly at stirrup locations. Diagonal cracks that lead to failure started to occur in wall web at the sixth cycle in negative direction. Failure of the specimen was sudden after occurrence of vertical splitting cracks in boundary element as well as crushing of concrete at the bottom of boundary element. The specimen's photograph after testing is displayed on Figure 4.24. Note that the numbers in the photo represent cycle number and direction on which the respective cracks occurred. Moreover, respective drift ratio and force are given as well. The maximum forces recorded were +892.29 kN and -1209.60 kN in positive and negative directions, respectively. The respective drifts at those forces were +0.39% and -0.79%.

Figure 4.23 Crack propagation stages for specimen J1.

Figure 4.24 Specimen J1 after testing.

The force-drift relationship for specimen J1 is displayed on Figure 4.25 while the envelope curve is displayed on Figure 4.26. From these figures, it can be seen clearly that in the positive loading direction, the test data is only available up to drift of +0.40% due to problem explained before. Thus, the complete envelope curve is only obtained for the negative loading direction. From the envelope curve, it can be seen that the wall stiffness starts to degrade at drift of 0.10% due to occurrence of cracks, both in positive and negative loading directions. Furthermore, before reaching the peak force, the hysteretic behaviour of the specimen looks like stress-strain curve of concrete in compression with no significant pinching behaviour observed. After reaching its peak point, the force drops significantly due to crushing of concrete in boundary elements and wall web.

Figure 4.25 Force-drift relationship of specimen J1.

Figure 4.26 Envelope curve of specimen J1.

Wall deformation components consist of flexure deformation, shear deformation, and sliding shear deformation which were measured using LVDTs in this experiment. The contribution of each deformation component to total wall drift for specimen J1 is displayed on Figure 4.27. It is worth noting that the summation of wall deformation components range from 80% to 120% of total wall drift. The source of error most probably due to the total wall drift was measured at height of 1200 mm from wall base (using DT1 and DT2) while measurement of wall deformation components (flexure and shear deformations) were only done up to height of 1000 mm from wall base and the deformation of top beam (at height of 1000 mm to 1200 mm from wall base) was assumed to be the same as the previous section. Nevertheless, for all specimens, when it is close to the peak forces, the summation of wall deformation components are very close to 100% of total wall drift. As can be seen in Figure 4.27, for specimen J1, shear deformation dominates while sliding

shear deformation is almost negligible. Furthermore, it can be seen that as the total wall drift increases, shear deformation also increases whereas flexure deformation decreases. After the peak force (at drift of -0.80%), shear deformation becomes very large and overestimated because of widening of main diagonal crack in wall web.

Figure 4.27 Contribution of wall deformation components to total wall drift of specimen J1.

Strain distributions in reinforcement bars are presented in terms of graphs with coordinate system of wall specimen to be one of the graph axes. The coordinate system of wall specimen is explained as follow. X is in the direction of wall length with coordinate of $x = 0$ is the middle point of wall. Thus, coordinate of $x = -500$ mm is the left end of wall (denotes by A in the strain gauges setup) while $x = 500$ mm is the right end of wall (denotes by D in the strain gauges setup). Y is in the direction of wall thickness or boundary element with coordinate of $y = 0$ is the middle

point of wall web. Hence, coordinate of $y = -250$ mm is the edge of south boundary element (denotes by S in the strain gauges setup) while $y = 250$ mm is the edge of north boundary element (denotes by N in the strain gauges setup). Z is in the direction of wall height with $z = 0$ is the base of wall. Therefore, coordinate of $z = 500$ mm is the mid-height of wall while $z = 1000$ mm is the top of wall for specimens J1-J4 whereas for specimens J5-J7, coordinate of $z = 1000$ mm is the mid-height of wall while $z = 2000$ mm is the top of wall. This coordinate system valid for all specimens tested in this study.

The strain distributions for specimen J1 are presented for negative loading direction only since it represents the complete envelope curve until the specimen failed. The strains are presented in several drift stages starting from occurrence of cracks until peak force. Firstly, strains in vertical bars are discussed. From the loading test setup, it can be seen that the specimen is a cantilever wall with lateral loading applied at the top of the specimen. Thus, the maximum bending moment at the wall cross section is at the wall base. By plotting the vertical strain profile at wall base, the strain distribution along wall cross section can be seen. The vertical strain profile at wall base is plotted at coordinate of $z = 50$ mm and is displayed on Figure 4.28. From the figure, it can be concluded that assumption plane section remains plane is not applicable here since the strain distribution is non-linear. This is because for wall with height to length ratio of 1.0, it behaves almost the same with deep beam, and thus assumption of linear strain distribution along cross section is not valid. It is worth noting that strain of vertical bar at coordinate of $x = 400$ mm is in tension (which actually supposed to be in compression) because the bar was crossed by a diagonal crack which started to occur at drift of -0.20%. Furthermore, it can be seen also from the figure that none of vertical bar at wall base has yielded during testing (all strain values are below the yield strain of the bars).

Figure 4.28 Strains of vertical bars at wall base ($z = 50$ mm).

However, in order to observe further whether there is any vertical bar has yielded during testing, the strain values in vertical bars are plotted at points where the strain gauges are closest to the major diagonal crack that lead to specimen failure. This plot is presented on Figure 4.29. From this figure, it can be seen that one of vertical bar (at $x = 0$) has yielded at drift of -0.80% which corresponds to point of maximum force.

Another observation that can be done in the Y direction of wall specimen is to see whether there is any variation of strain values in vertical bars in boundary elements. The strain profile is shown on Figure 4.30 for vertical bars located at bottom left end and middle left end of wall which are subjected to maximum tension. From the figure, it can be concluded that there is almost no difference in strains between vertical bars which are located close to wall web with those that located far away

from wall web. This means that all vertical bars are equally effective and flange width of 500 mm is fully effective for wall with height of 1000 mm.

Figure 4.29 Strains of vertical bars at points nearest to major diagonal crack.

For strains in horizontal bars, they are given at points where strain gauges located nearest to major diagonal crack in order to observe whether the bars have yielded during testing. The plot is presented on Figure 4.31. From the figure, it can be seen that almost all horizontal bars yielded during testing and the average strain values is much higher than that of vertical bars. This means that for specimen J1, horizontal bars are more effective than vertical bars in contributing to the overall shear strength of the specimen.

Figure 4.30 Strains of vertical bars at bottom left end and middle left end of the wall ($x = -462$ mm; $z = 50$ mm and 450 mm).

Another observation of horizontal bars strains is done for stirrups in the boundary elements. Based on numerical data, the average strain values of stirrups in boundary elements is very low and insignificant. Thus, the strain values are not plotted here. This means that the stirrups in boundary elements do not contribute much to the overall shear strength of the specimen.

Figure 4.31 Strains of horizontal bars at points nearest to major diagonal crack.

4.6.2 Specimen J2

For specimen J2, concrete cylinder compressive strength at the day of testing was 96.8 MPa. There was no problem observed during testing of specimen J2 as in the case of specimen J1, so complete data for force-drift relationship and envelope curve was obtained for both positive and negative loading directions. Crack propagation stages of specimen J2 during testing are displayed on Figure 4.32 while photo of the specimen after testing is displayed on Figure 4.33. Furthermore, force-drift relationship and envelope curve are displayed on Figures 4.34 and 4.35, respectively.

Figure 4.32 Crack propagation stages for specimen J2.

Similar as in the case of specimen J1, for specimen J2, diagonal cracks started to occur in wall web as early as in the second cycle and the number of cracks increased in subsequent cycles. In boundary element flange, horizontal cracks started to occur in the third and fourth cycles and they were located mostly at stirrup locations. At the eight cycle, the number of diagonal cracks were already so many and they filled up the wall web. At the ninth cycle, in negative loading direction, web crushing was observed together with vertical splitting cracks at boundary element flange and the force dropped significantly. The maximum forces recorded

during testing were +1264.75 kN and -1270.82 kN in positive and negative loading directions, respectively. The respective drifts at those forces were +0.80% and -0.68%.

Figure 4.33 Specimen J2 after testing.

Like the case of specimen J1, the force-drift relationship of specimen J2 is also similar to stress-strain curve of concrete in compression with no significant pinching behaviour observed. At the ninth cycle in negative loading direction, the force could not go back to its previous state in previous cycle. Instead, it dropped after reaching previous drift amplitude of -0.70% together with occurrence of web crushing and vertical splitting cracks in boundary element flange. Thus, it can be concluded that the specimen has already failed in this cycle and the testing was no longer

continued for the next cycle. From the envelope curve, it is worth noting that in positive loading direction, the wall stiffness degrades gradually as the drift increases while in negative loading direction, it can be seen clearly that the wall stiffness starts to degrade at drift of -0.10% due to occurrence of diagonal cracks.

Figure 4.34 Force-drift relationship of specimen J2.

Wall deformation components contribution to total wall drift for specimen J2 is displayed on Figure 4.36. It is worth noting that the summation of wall deformation components range from 80% to 120% of total wall drift. Like in the case of specimen J1, the source of error is most probably due to the total wall drift was measured at height of 1200 mm from wall base (using DT1 and DT2) while measurement of wall deformation components (flexure and shear deformations) were only done up to height of 1000 mm from wall base and the deformation of top beam (at height of 1000 mm to 1200 mm from wall base) was assumed to

be the same as the previous section. Nevertheless, when it is close to the peak forces, the summation of wall deformation components are very close to 100% of total wall drift. As can be seen in Figure 4.36, for specimen J2, shear deformation dominates while sliding shear deformation is very small. Furthermore, the same phenomenon observed in specimen J1 also happened here for specimen J2 that as the total wall drift increases, shear deformation also increases whereas flexure deformation decreases. In addition, after the peak force in negative direction (at drift of -0.68%), shear deformation becomes very large and overestimated because of widening of diagonal cracks in wall web while in positive direction, the same thing cannot be observed since the envelope curve shows no degradation yet in lateral force until drift of +0.80% and the testing was no longer continued for the next cycle.

Figure 4.35 Envelope curve of specimen J2.

Figure 4.36 Contribution of wall deformation components to total wall drift of specimen J2.

The strain distributions for specimen J2 are presented for negative loading direction only since the first strength degradation was observed and the specimen has failed in this direction as well. The strains are presented in several drift stages starting from occurrence of cracks until peak force. First, strains in vertical bars are discussed. Like in the case of specimen J1, the vertical strain profile at wall base is plotted at coordinate of $z = 50$ mm and is displayed on Figure 4.37. It should be noted that strain gauge at coordinate of $x = -200$ mm was not functioning at all during testing, and hence there was no data recorded. From the figure, it can be seen that at drift of -0.10% , the linear strain distribution over wall cross section is observed, whereas after that, the strain distributions are quite irregular and it is difficult to make any conclusion from the readings. It is worth noting that none of vertical bar at wall base has

yielded during testing (all strain values are below the yield strain of the bars).

Figure 4.37 Strains of vertical bars at wall base ($z = 50$ mm).

Like in the case of specimen J1, for specimen J2, strain values in vertical bars are also plotted at points where the strain gauges are located nearest to major diagonal cracks. This plot is presented on Figure 4.38. From the figure, it can be seen that many of vertical bars are in high tension with one of them (located at $x = 200$ mm) has yielded at drift of -0.80%.

Another observation is done in the Y direction of wall specimen to see whether there is any variation of strain values in vertical bars in boundary elements. The strain profile is shown on Figure 4.39 for vertical bars located at bottom left end and middle left end of wall which are subjected

to maximum tension. From the figure, it can be seen that in average, there is little difference of strain values between vertical bars which are located close to wall web with those that are located far away from wall web, except for bars that are located at $z = 50$ mm; $y = 132$ mm and 212 mm, in which it is observed that the strain values decrease as the distance of the bars to wall web increases. However, this phenomenon is not observed for bars located at $z = 50$ mm; $y = -212$ mm, in which the strain values are similar to the bars located adjacent to wall web.

Figure 4.38 Strains of vertical bars at points nearest to major diagonal crack.

For strains in horizontal bars, they are given at points where strain gauges located nearest to major diagonal crack in order to observe whether the horizontal bars have yielded during testing. The plot is displayed on Figure 4.40. From the figure, it can be seen that only one of

Figure 4.39 Strains of vertical bars at bottom left end and middle left end of the wall ($x = -462$ mm; $z = 50$ mm and 450 mm).

Figure 4.40 Strains of horizontal bars at points nearest to major diagonal crack.

4.6.3 Specimen J3

For specimen J3, concrete cylinder compressive strength at the day of testing was 110.7 MPa. There was no problem observed during testing of specimen J3 as in the case of specimen J1, so complete data for force-drift relationship and envelope curve was obtained for both positive and negative loading directions. Crack propagation stages of specimen J3 during testing are displayed on Figure 4.41 while photo of the specimen after testing is displayed on Figure 4.42. Furthermore, force-drift relationship and envelope curve are displayed on Figures 4.43 and 4.44, respectively.

Figure 4.41 Crack propagation stages for specimen J3.

Similar to the case of specimens J1 and J2, for specimen J3, diagonal cracks started to occur in wall web as early as in the second cycle and the number of cracks increased in subsequent cycles. In boundary element flange, horizontal cracks started to occur in the second and third cycles and they were located mostly at stirrup locations. At the seventh cycle, the number of diagonal cracks were already so many and they filled up the wall web. Similar to specimen J2, at the ninth cycle, in negative loading direction, web crushing was observed together with vertical splitting cracks at boundary element flange and the lateral force dropped

can be seen that starting from drift of 0.10%, the wall stiffness starts to degrade with occurrence of diagonal cracks in wall web.

Figure 4.43 Force-drift relationship of specimen J3.

Wall deformation components contribution to total wall drift for specimen J3 is displayed on Figure 4.45. It is worth noting that the summation of wall deformation components range from 80% to 110% of total wall drift. Like in the case of specimens J1 and J2, the source of error most probably due to the total wall drift was measured at height of 1200 mm from wall base (using DT1 and DT2) while measurement of wall deformation components (flexure and shear deformations) were only done up to height of 1000 mm from wall base and the deformation of top beam (at height of 1000 mm to 1200 mm from wall base) was assumed to be the same as the previous section. Nevertheless, when it is close to the peak forces, the summation of wall deformation components are very close to 100% of total wall drift. As can be seen in Figure 4.45, for

specimen J3, in early stage (drift of 0.10%), flexure deformation still dominates whereas starting from drift of 0.20% onwards, shear deformation dominates while sliding shear deformation is very small at all times. Furthermore, the same phenomenon observed in specimens J1 and J2 also happened here for specimen J3 that as the total wall drift increases, shear deformation also increases whereas flexure deformation decreases. However, after the peak force in negative direction (at drift of -0.76%), shear deformation becomes very large but not overestimated like in the case of specimens J1 and J2 since for specimen J3, the summation of wall deformation components is only about 94% of total wall drift.

Figure 4.44 Envelope curve of specimen J3.

Figure 4.45 Contribution of wall deformation components to total wall drift of specimen J3.

The strain distributions for specimen J3 are presented for negative loading direction only since the specimen failed in this direction. The strains are presented in several drift stages starting from occurrence of cracks until peak force. Firstly, strains in vertical bars are discussed. Like in the case of specimens J1 and J2, the vertical strain profile at wall base is plotted at coordinate of $z = 50$ mm and is displayed on Figure 4.46. It should be noted that strain gauge at coordinate of $x = -200$ mm was not functioning at all during testing, and hence there was no data recorded. From the figure, it can be seen that at drift of -0.10% and -0.20% , the linear strain distribution over wall cross section can still be observed, whereas after that, the strain distributions are quite irregular and it is difficult to make any conclusion from the readings. Nevertheless, it is worth noting that none of vertical bar at wall base has yielded during testing (all strain values are below the yield strain of the bars).

Figure 4.46 Strains of vertical bars at wall base ($z = 50$ mm).

Like in the case of specimens J1 and J2, for specimen J3, strain values in vertical bars are also plotted at points where the strain gauges are located nearest to major diagonal cracks. This plot is presented on Figure 4.47. It should be noted, however, that the strain gauges at coordinate of $x = -200$ mm (whether located at bottom, middle, and top of wall) were not functioning at all during testing, and thus no data was captured during testing for strain in vertical bar at $x = -200$ mm. From the figure, it can be seen that none of vertical bar has yielded during testing for specimen J3. This finding is quite different with that of specimens J1 and J2 where at least there is one vertical bar yielded during testing.

Another observation is done in the Y direction of wall specimen to see whether there is any variation of strain values in vertical bars in boundary

elements. The strain profile is shown on Figure 4.48 for vertical bars located at bottom left end and middle left end of wall. It should be noted that strain gauge located at $y = -212$ mm; $z = 50$ mm was not functioning at all during testing and hence no data was recorded for this particular coordinate. From the figure, it is interesting to note that the strain values for vertical bars that are close to wall web are higher than those located far away from wall web. This finding for specimen J3 is again quite different with those observed in specimens J1 and J2 where in average, the strain values are similar between those vertical bars in boundary elements.

Figure 4.47 Strains of vertical bars at points nearest to major diagonal crack.

Figure 4.48 Strains of vertical bars at bottom left end and middle left end of the wall ($x = -462$ mm; $z = 50$ mm and 450 mm).

For strains in horizontal bars, they are given at points where strain gauges located nearest to major diagonal crack in order to observe whether the horizontal bars have yielded during testing. The plot is displayed on Figure 4.49. From the figure, it can be seen that many of horizontal bars were in high tension and two of them yielded during testing (located at $z = 200$ mm and 600 mm). Furthermore, the average strain values is higher than that of vertical bars. This means that for specimen J3, horizontal bars are more effective than vertical bars in contributing to the overall shear strength of the specimen. This finding is similar with that observed in specimen J1 but opposite with that observed in specimen J2. Furthermore, for stirrups in boundary elements, similar to the case of specimens J1 and J2, the average strain values is quite low and

insignificant. Hence, no plot is given here. This means that the stirrups in boundary elements do not contribute much to the overall shear strength of the specimen.

Figure 4.49 Strains of horizontal bars at points nearest to major diagonal crack.

4.6.4 Specimen J4

For specimen J4, concrete cylinder compressive strength at the day of testing was 93.5 MPa. For this specimen, after reaching peak point in the positive direction, there was strength degradation for subsequent cycles. At the ninth cycle (last cycle), the drift amplitude was supposed to be increased up to +0.80%. However, at drift of +0.70%, the strength of the specimen was almost half of the peak point. Thus, further positive drift increment was aborted to prevent severe strength degradation and the

testing was continued to fail the specimen in negative direction. Crack propagation stages of specimen J4 during testing are displayed on Figure 4.50 while photo of the specimen after testing is displayed on Figure 4.51. Furthermore, force-drift relationship and envelope curve are displayed on Figures 4.52 and 4.53, respectively.

Figure 4.50 Crack propagation stages for specimen J4.

The maximum forces recorded during testing were +810.74 kN and - 826.12 kN in positive and negative directions, respectively. The respective drifts at maximum forces were +0.54% and -0.73%. Diagonal cracks started to occur at third cycle together with horizontal cracks at stirrup locations of edge column and number of cracks increased in subsequent cycles. Failure of specimen happened when concrete at the bottom of edge column crushed. Furthermore, web crushing was also spotted and horizontal web reinforcement fracture was also observed at the middle of wall web where concrete spalled.

For specimen J4, although in positive and negative directions the crack patterns are quite similar with crushing of the bottom edge column observed in both directions and the maximum forces are also similar, the force-drift relationship and envelope curve show that there is slightly different behaviour observed in positive and negative directions. Even

though wall stiffness starts to degrade at drift of 0.10% in both directions, the stiffness degradation is more severe for negative direction. Hence, the specimen reached its peak point in positive direction first at drift of +0.54% as compared to negative direction which reached its peak point at drift of -0.73%. Moreover, in positive direction, after reaching its peak point, the force dropped significantly in subsequent cycles whereas for negative direction, the peak force could be maintained for the subsequent cycle until drift of -0.90% before it dropped significantly.

Figure 4.51 Specimen J4 after testing.

Figure 4.52 Force-drift relationship of specimen J4.

Wall deformation components contribution to total wall drift for specimen J4 is displayed on Figure 4.54. It is worth noting that the summation of wall deformation components range from 70% to 120% of total wall drift. Like previous specimens, the source of error most probably due to the total wall drift was measured at height of 1200 mm from wall base (using DT1 and DT2) while measurement of wall deformation components (flexure and shear deformations) were only done up to height of 1000 mm from wall base and the deformation of top beam (at height of 1000 mm to 1200 mm from wall base) was assumed to be the same as the previous section. Nevertheless, when it is close to the peak forces, the summation of wall deformation components are very close to 100% of total wall drift. As can be seen in Figure 4.54, for specimen J4, different behaviour is observed in positive and negative directions. In positive direction, flexure deformation dominates for the whole drift range whereas in negative direction, in early stage up to drift

of -0.60%, flexure deformation still dominates while after that shear deformation dominates. This is quite different with specimens J1, J2, and J3 where shear deformation always dominates. This may be due to the flexural rigidity of specimen J4 without boundary elements is less than the flexural rigidity of specimens J1, J2, and J3 with boundary elements. Hence, there was tendency that flexure deformation could dominate although the wall itself ultimately failed in shear. Furthermore, it is worth noting that for specimen J4 without boundary elements, after the peak points reached (drift of +0.54% and -0.73%), sliding shear deformation starts to give significant contribution to total wall drift. This phenomenon did not happen in specimens with boundary elements (J1, J2, and J3) where sliding shear deformation is very small and thus it is negligible. This finding is in agreement with experiment conducted by Corley et al. (1981) who concluded that boundary elements could help to prevent sliding shear failure by providing stiff dowels.

Figure 4.53 Envelope curve of specimen J4.

Figure 4.54 Contribution of wall deformation components to total wall drift of specimen J4.

The strain distributions for specimen J4 are presented for positive direction only since the first strength degradation was observed in this direction. The strains are presented in several drift stages starting from occurrence of cracks until peak force. Firstly, strains in vertical bars are discussed. Like previous specimens, the vertical strain profile at wall base is plotted at coordinate of $z = 50$ mm and is displayed on Figure 4.55. From the figure, it can be seen that at drift of +0.10%, the linear strain distribution over wall cross section can still be observed, whereas after that, the strain distributions are non-linear showing bending behaviour with some exception of strains at several points where the cracks crossed. This bending behaviour observed may be due to the fact that flexure deformation dominates for specimen J4 in positive direction. This finding is quite different with those of specimens J1, J2, and J3 where the strain

distributions are irregular because of shear deformation dominates in those specimens. It is worth noting that one of vertical bars at wall base has yielded during testing (coordinate at $x = -462$ mm). This may be due to concrete crushing at this particular location (bottom of edge column).

Figure 4.55 Strains of vertical bars at wall base ($z = 50$ mm).

Like previous specimens, strain values in vertical bars are also plotted at points where the strain gauges are located nearest to major diagonal cracks. This plot is presented on Figure 4.56. It should be noted, however, that the strain gauge at coordinate of $x = 0$ mm was not functioning at all during testing, and thus no data was captured. From the figure, it can be seen that one of vertical bars has yielded during testing for specimen J4 (at coordinate of $x = -462$ mm) which is already discussed previously.

Figure 4.56 Strains of vertical bars at points nearest to major diagonal crack.

For strains in horizontal bars, they are given at points where strain gauges located nearest to major diagonal crack in order to observe whether the bars have yielded during testing. The plot is presented on Figure 4.57. From the figure, it can be seen that almost all horizontal bars yielded during testing. It is worth noting that one of horizontal bars (at coordinate of $z = 400$ mm) has fractured at drift of +0.54% and thus there was no more strain readings from this drift onwards. Furthermore, the average strain values of horizontal bars is much higher than that of vertical bars. This means that for specimen J4, horizontal bars are more effective than vertical bars in contributing to the overall shear strength of the specimen.

Another observation of horizontal bars strains is done for stirrups in the edge columns. Based on numerical data, the average strain values of

stirrups in edge columns is very low and insignificant. Thus, the strain values are not plotted here. This means that the stirrups in the edge columns do not contribute much to the overall shear strength of the specimen.

Figure 4.57 Strains of horizontal bars at points nearest to major diagonal crack.

4.6.5 Specimen J5

Similar case to specimen J1, concrete cylinder compressive strength for specimen J5 was taken from cylinder compressive strength tested at the age of 91 days although the specimen J5 itself was tested at a later date due to unavailable data of cylinder compressive strength at the day of specimen testing. However, since concrete usually reaches its optimum strength at the age of 91 days, thus it can be assumed that the

compressive strength development after 91 days is negligible. Based on the test at the age of 91 days, the concrete cylinder compressive strength for specimen J5 was 103.3 MPa. Crack propagation stages of specimen J5 during testing are displayed on Figure 4.58 while photo of the specimen after testing is displayed on Figure 4.59. Furthermore, force-drift relationship and envelope curve are displayed on Figures 4.60 and 4.61, respectively.

For specimen J5, small diagonal cracks in bottom right of wall web and horizontal cracks at bottom right of boundary element flange started to occur as early as in the second cycle in positive direction. However, in negative direction, there were no cracks observed at the second cycle. Instead, cracks started to occur at the third cycle for negative direction. Number of cracks increased in subsequent cycles. At the eight cycle, major diagonal cracks in positive and negative directions started to open quite wide as can be seen in Figure 4.58. It is worth noting that at this cycle, the maximum forces during testing were observed for both directions. Web crushing started to occur in the tenth cycle, before the specimen failed. Failure of the specimen happened in the positive direction with occurrence of vertical splitting cracks in the boundary element flange and crushing of the bottom left end of boundary element flange. However, this phenomenon could only be observed for positive direction. In negative direction, vertical splitting cracks and crushing of the boundary element flange were not as severe as in the positive direction even though at later stage of testing, the drift level was increased to -1.60% in negative direction as compared to +1.20% in positive direction. The maximum forces recorded during testing were +595.76 kN and -556.97 kN in positive and negative loading directions, respectively. The respective drifts at those forces were +0.70% and -0.70%.

Figure 4.58 Crack propagation stages for specimen J5.

Unlike previous lower specimens, the force-drift relationship of specimen J5 which has higher height to length ratio shows more ductile behaviour as compared to previous specimens. After reaching peak forces at the eight cycle, the lateral forces did not drop significantly at subsequent cycles. Instead, the forces could be maintained up to two subsequent cycles before they started to drop significantly. Furthermore, pinching behaviour could be observed starting from ninth cycle onwards. The envelope curve shows that in positive direction, the lateral force dropped significantly at drift level of +0.95% together with occurrence of

vertical splitting cracks and crushing of bottom left end of boundary element flange whereas in negative direction, the lateral force dropped gradually starting from drift level of -0.90% onwards with less severe cracks and crushing at boundary element flange as compared to positive direction. Moreover, from the envelope curve, it is worth noting that in both directions, the wall stiffness starts to degrade at drift level of 0.10% although in negative direction, there were no visible cracks observed yet at this point.

Figure 4.59 Specimen J5 after testing.

Figure 4.60 Force-drift relationship of specimen J5.

Wall deformation components contribution to total wall drift for specimen J5 is displayed on Figure 4.62. It is worth noting that the summation of wall deformation components range from 70% to 130% of total wall drift. The source of error is most probably due to the total wall drift was measured at height of 2200 mm from wall base (using DT1 and DT2) while measurement of wall deformation components (flexure and shear deformations) were only done up to height of 2000 mm from wall base and the deformation of top beam (at height of 2000 mm to 2200 mm from wall base) was assumed to be the same as the previous section. Nevertheless, when it is close to the peak forces, the summation of wall deformation components are close to 100% of total wall drift. As can be seen in Figure 4.62, for specimen J5, similar behaviour was observed in positive and negative directions. In early stage up to drift level of 0.70% (point of maximum forces), flexure deformation still dominates the

overall behaviour. However, starting from drift level of 0.70% onwards where major diagonal cracks started to open quite wide, shear deformation becomes more dominant. The shear deformation is overestimated in negative direction starting from drift level of -0.90% onwards where the lateral force started to drop gradually. Nevertheless, the same phenomenon was not observed in positive direction where the total deformation components is close to 100% of total wall drift at drift level of +0.95% onwards. It is worth noting that in both directions, as the drift increases, contribution of flexure deformation decreases whereas contribution of shear deformation increases. In addition, in all stages, the sliding shear deformation contribution is very small and thus it is negligible.

Figure 4.61 Envelope curve of specimen J5.

Figure 4.62 Contribution of wall deformation components to total wall drift of specimen J5.

The strain distributions for specimen J5 are presented for positive direction only since the first strength degradation was observed in this direction. The strains are presented in several drift stages starting from occurrence of cracks until peak force. Firstly, strains in vertical bars are discussed. Like previous specimens, the vertical strain profile at wall base is plotted at coordinate of $z = 50$ mm and is displayed on Figure 4.63. It should be noted that strain gauge located at $x = 200$ mm was not functioning at all during testing, and thus no data has been recorded at this particular coordinate. Moreover, strain gauge located at $x = -200$ mm was only functioning up to drift of $+0.10\%$ while after that no data has been recorded at this particular coordinate. From the figure, it can be seen that at drift of $+0.10\%$, the linear strain distribution over wall cross section can still be observed, whereas after that, the strain distributions are irregular with high tensile strains observed at several points where the

cracks crossed. It is worth noting that none of vertical bars at wall base has yielded during testing.

Figure 4.63 Strains of vertical bars at wall base ($z = 50$ mm).

Like previous specimens, strain values in vertical bars are also plotted at points where the strain gauges are located nearest to major diagonal cracks. This plot is presented on Figure 4.64. From the figure, it can be seen that one of vertical bars has yielded during testing for specimen J5 (at coordinate of $x = 0$ mm) which is located in the middle of wall web where major diagonal cracks for positive and negative directions crossed each other. This is also the point where web crushing was observed.

Another observation is done in the Y direction of wall specimen to see whether there is any variation of strain values in vertical bars in boundary elements. The strain profile is shown on Figure 4.65 for vertical bars

located at bottom right end and middle right end of wall which are subjected to maximum tension. From the figure, it can be seen that in average, there is little difference of strain values between vertical bars which are located close to wall web with those that are located far away from wall web, except for bars that are located at bottom right end of wall ($z = 50$ mm) in positive coordinate of Y axis.

Figure 4.64 Strains of vertical bars at points nearest to major diagonal crack.

For strains in horizontal bars, they are given at points where strain gauges located nearest to major diagonal crack in order to observe whether the bars have yielded during testing. The plot is presented on Figure 4.66. From the figure, it can be seen that half of horizontal bars yielded during testing. Furthermore, average strain values of horizontal bars is higher than that of vertical bars. This means that for specimen J5,

horizontal bars are more effective than vertical bars in contributing to the overall shear strength of the specimen.

Another observation of horizontal bars strains is done for stirrups in the boundary elements. Based on numerical data, the average strain values of stirrups in boundary elements is very low and insignificant. Hence, the strain values are not plotted here. This means that the stirrups in the boundary elements do not contribute much to the overall shear strength of the specimen.

Figure 4.65 Strains of vertical bars at bottom right end and middle right end of the wall ($x = 462$ mm; $z = 50$ mm and 950 mm).

Figure 4.66 Strains of horizontal bars at points nearest to major diagonal crack.

4.6.6 Specimen J6

For specimen J6, since concrete cylinder compressive strength at the day of testing was not available, the concrete cylinder compressive strength was taken the same as specimen J2, i.e. 96.8 MPa since they were cast together. Furthermore, specimen J6 was tested at a later date than specimen J2 and both of them were tested at age more than 91 days, and hence it could be assumed that concrete strength development between day of testing of specimen J2 and specimen J6 was negligible. Crack propagation stages of specimen J6 during testing are displayed on Figure 4.67 while photo of the specimen after testing is displayed on Figure 4.68. Furthermore, force-drift relationship and envelope curve are displayed on Figures 4.69 and 4.70, respectively.

Figure 4.67 Crack propagation stages for specimen J6.

The crack patterns of specimen J6 were quite similar to those of specimen J5. Small diagonal cracks in wall web and horizontal cracks at boundary element flange started to occur as early as in the second cycle in both positive and negative directions and the number of cracks increased in subsequent cycles. At the sixth cycle, major diagonal cracks in positive and negative directions have already occurred and they started to open quite wide in the ninth cycle as can be seen in Figure 4.67. The lateral force started to drop significantly at the tenth cycle in negative direction together with occurrence of vertical splitting cracks and

crushing at the bottom of boundary element flange. The same phenomenon also happened in positive direction at the next cycle which can be seen in Figure 4.68 for the specimen photo after testing. The maximum forces recorded during testing were +724.14 kN and -673.00 kN in positive and negative loading directions, respectively. The respective drifts at those forces were +0.80% and -0.71%.

Figure 4.68 Specimen J6 after testing.

Figure 4.69 Force-drift relationship of specimen J6.

The force-drift relationship of specimen J6 shows that it has more brittle behaviour as compared to specimen J5. In specimen J6, the lateral forces dropped gradually in the subsequent cycle after reaching their peak points and they also dropped significantly in two subsequent cycles. On the other hand, in specimen J5, the lateral forces could be maintained up to two subsequent cycles before they started to drop significantly. For specimen J6, pinching behaviour could be observed starting from the tenth cycle onwards after first strength degradation observed at the end of ninth cycle (drift level of -0.80%). This was when the major diagonal crack started to open very wide. Moreover, from the envelope curve, it is worth noting that in positive direction, the wall stiffness starts to degrade gradually starting from drift level of +0.05% although there are no visible cracks observed yet in this point. On the other hand, in negative direction, the wall stiffness starts to degrade at drift level of -0.10% where cracks start to occur.

Figure 4.70 Envelope curve of specimen J6.

Wall deformation components contribution to total wall drift for specimen J6 is displayed on Figure 4.71. It is worth noting that the summation of wall deformation components range from 80% to 110% of total wall drift. Like in the case of specimen J5, the source of error most probably due to the total wall drift was measured at height of 2200 mm from wall base (using DT1 and DT2) while measurement of wall deformation components (flexure and shear deformations) were only done up to height of 2000 mm from wall base and the deformation of top beam (at height of 2000 mm to 2200 mm from wall base) was assumed to be the same as the previous section. Nevertheless, when it is close to the peak forces, the summation of wall deformation components are very close to 100% of total wall drift. As can be seen in Figure 4.71, similar to the case of specimen J5, for specimen J6, in early stage up to drift level of +0.80% and -0.71% (point of maximum forces), flexure deformation still

dominates the overall behaviour. However, starting from drift level of +0.80% and -0.71% onwards where major diagonal cracks started to open very wide, shear deformation becomes the dominant one. Furthermore, it is worth noting that in both directions, as the drift increases, contribution of flexure deformation decreases whereas contribution of shear deformation increases. In addition, in all stages, sliding shear deformation contribution is very small and hence it can be neglected. These two observations are observed in specimen J5 as well.

Figure 4.71 Contribution of wall deformation components to total wall drift of specimen J6.

The strain distributions for specimen J6 are presented for positive direction only since the maximum strength of the specimen before first strength degradation was observed in this direction. The strains are presented in several drift stages starting from occurrence of cracks until peak force. Firstly, strains in vertical bars are discussed. Like previous

specimens, the vertical strain profile at wall base is plotted at coordinate of $z = 50$ mm and is displayed on Figure 4.72. From the figure, it can be seen that at drift of $+0.10\%$, the linear strain distribution over wall cross section can still be observed, with the exception for strain gauge at coordinate of $x = 200$ mm which is located near to the first diagonal cracks observed and therefore it has higher strain value as compared to other strain gauges. For specimen J6, it still can be seen regular pattern of non-linear strain distribution with tensile strains at the right end of the wall while compression strains at the left end of the wall. It is worth noting that none of vertical bars at wall base has yielded during testing.

Figure 4.72 Strains of vertical bars at wall base ($z = 50$ mm).

Figure 4.73 Strains of vertical bars at points nearest to major diagonal crack.

Like previous specimens, strain values in vertical bars are also plotted at points where the strain gauges are located nearest to major diagonal cracks. This plot is presented on Figure 4.73. It should be noted that one of strain gauges (at coordinate of $x = -200$ mm) which is located near to major diagonal crack was not functioning at all during testing, and therefore no data has been recorded at that particular coordinate. From the figure, it can be seen that none of vertical bars has yielded during testing for specimen J6. Highest strain value observed is in the middle of wall (at $x = 0$ mm) where major diagonal cracks crossed each other. However, the strain value shows that the bar has not yielded yet which is different from specimen J5 in which the bar located at the same location has yielded during testing. It should be noted that the vertical web bars

used in specimen J6 have bigger size than those used in specimen J5 (D10 vs D6) which may be the cause of this.

Figure 4.74 Strains of vertical bars at bottom right end and middle right end of the wall ($x = 462$ mm; $z = 50$ mm and 950 mm).

Another observation is done in the Y direction to see any variation of strain values in vertical bars in boundary elements. The strain profile is shown on Figure 4.74 for vertical bars located at bottom right end and middle right end of wall. From the figure, it can be seen that for $z = 50$ mm, strain values of vertical bars located close to wall web are smaller than those that are located far away from wall web. This finding is quite different with other specimens where in other specimens, it is observed that bars located close to wall web generally have similar or even higher strain values as compared to bars located far away from wall web.

Figure 4.75 Strains of horizontal bars at points nearest to major diagonal crack.

For strains in horizontal bars, they are given at points where strain gauges located nearest to major diagonal crack in order to observe whether the bars have yielded during testing. The plot is presented on Figure 4.75. From the figure, it can be seen that almost all horizontal bars yielded during testing. Furthermore, average strain values of horizontal bars is much higher than that of vertical bars. This means that for specimen J6, horizontal bars are more effective than vertical bars in contributing to the overall shear strength of the specimen which is also observed in specimen J5.

Another observation of horizontal bars strains is done for stirrups in the boundary elements. Based on numerical data, the average strain

values of stirrups in boundary elements is very low and insignificant. Therefore, the strain values are not plotted here. This means that the stirrups in the boundary elements do not contribute much to the overall shear strength of the specimen.

4.6.7 Specimen J7

For specimen J7, since concrete cylinder compressive strength at the day of testing was not available, the concrete cylinder compressive strength was taken the same as specimen J3, i.e. 110.7 MPa since they were cast together. Furthermore, specimen J7 was tested at a later date than specimen J3 and both of them were tested at age more than 91 days, and hence it could be assumed that concrete strength development between day of testing of specimen J3 and specimen J7 was negligible. Crack propagation stages of specimen J7 during testing are displayed on Figure 4.76 while photo of the specimen after testing is displayed on Figure 4.77. Furthermore, force-drift relationship and envelope curve are displayed on Figures 4.78 and 4.79, respectively.

The crack patterns of specimen J7 were quite different to those of specimens J5 and J6. As can be seen in Figure 4.77, there is no single major diagonal crack as well as no vertical splitting cracks at boundary element flanges for specimen J7 as observed in specimens J5 and J6. For specimen J7, small diagonal cracks in wall web started to occur as early as in the second cycle in both loading directions and the number of cracks increased in subsequent cycles. Horizontal cracks at boundary element flange started to occur from the third cycle onwards. At the tenth cycle, web crushing and crushing of bottom left end of boundary element flange started to occur. At the twelfth cycle, web crushing became more apparent and crushing of bottom right end of boundary element flange started to take place. The maximum forces recorded during testing were +894.77 kN and -854.02 kN in positive and negative loading directions, respectively. The respective drifts at those forces were +1.17% and -0.99%.

Figure 4.76 Crack propagation stages for specimen J7.

The force-drift relationship of specimen J7 shows typical specimen failing in web crushing mode. In positive direction, the lateral force dropped significantly in subsequent cycle after reaching its peak point whereas in negative direction, the lateral force could be maintained in subsequent cycle before it dropped significantly in two subsequent cycles. It is worth noting that the specimen reached its maximum forces at drift level of +1.17% and -0.99% for positive and negative directions, respectively which are higher than specimens J5 and J6. For specimen J7, pinching behaviour could only be observed at the last cycle after first

Figure 4.78 Force-drift relationship of specimen J7.

Wall deformation components contribution to total wall drift for specimen J7 is displayed on Figure 4.80. It is worth noting that the summation of wall deformation components range from 90% to 140% of total wall drift. Like in the case of specimens J5 and J6, the source of error most probably due to the total wall drift was measured at height of 2200 mm from wall base (using DT1 and DT2) while measurement of wall deformation components (flexure and shear deformations) were only done up to height of 2000 mm from wall base and the deformation of top beam (at height of 2000 mm to 2200 mm from wall base) was assumed to be the same as the previous section. Nevertheless, when it is close to the peak forces, the summation of wall deformation components are very close to 100% of total wall drift. As can be seen in Figure 4.80, similar to the case of specimens J5 and J6, for specimen J7, in early stage up to drift level of +1.17% and -0.99% (point of maximum forces), flexure deformation still dominates the overall behaviour. However, starting from

drift level of +1.17% and -0.99% onwards, shear deformation becomes the dominant one. Furthermore, the shear deformation becomes overestimated when the wall strength starts to degrade at drift level of -1.20% onwards. The same phenomenon observed in specimens J5 and J6 also happened here that in both directions, as the drift increases, contribution of flexure deformation decreases whereas contribution of shear deformation increases. In addition, in all stages, sliding shear deformation contribution is very small and hence it can be neglected. Therefore, it can be concluded that boundary elements can help to prevent sliding shear failure. This finding is in agreement with experiment conducted by Corley et al. (1981).

Figure 4.79 Envelope curve of specimen J7.

Figure 4.80 Contribution of wall deformation components to total wall drift of specimen J7.

The strain distributions for specimen J7 are presented for positive direction only since the maximum strength of the specimen before first strength degradation was observed in this direction. The strains are presented in several drift stages starting from occurrence of cracks until peak force. Firstly, strains in vertical bars are discussed. Like previous specimens, the vertical strain profile at wall base is plotted at coordinate of $z = 50$ mm and is displayed on Figure 4.81. From the figure, it can be seen that at drift of +0.10%, the linear strain distribution over wall cross section can still be observed. As in the case of specimen J6, for specimen J7, it still can be seen regular pattern of non-linear strain distribution with tensile strains at the right end of the wall while compression strains at the left end of the wall up to drift level of +0.60% while after that, the strain distributions are irregular due to number of diagonal cracks that occur in

wall web increases. It is worth noting that none of vertical bars at wall base has yielded during testing.

Figure 4.81 Strains of vertical bars at wall base ($z = 50$ mm).

Like previous specimens, strain values in vertical bars are also plotted at points where the strain gauges are located nearest to major diagonal cracks. However, since in specimen J7, there is no major diagonal cracks as observed in specimens J5 and J6, hence the points plotted here are taken at the same coordinate as per specimens J5 and J6 in order to make consistent comparison. The plot is presented on Figure 4.82. From the figure, similar to specimen J6, it can be seen that none of vertical bars has yielded during testing for specimen J7. In addition, the average strain values is quite low which means that contribution of vertical web bars to the overall shear strength is less effective in specimen J7.

Figure 4.82 Strains of vertical bars at points nearest to major diagonal crack.

Another observation is done in the Y direction of wall specimen to see whether there is any variation of strain values in vertical bars in boundary elements. The strain profile is shown on Figure 4.83 for vertical bars located at bottom right end and middle right end of wall which are subjected to maximum tension. From the figure, it can be seen that up to drift level of +0.30%, in average, there is little difference of strain values between vertical bars which are located close to wall web with those that are located far away from wall web. However, starting from drift level of +0.60% onwards, the strain distributions are quite irregular and it is difficult to make any conclusions from the strain values.

Figure 4.83 Strains of vertical bars at bottom right end and middle right end of the wall ($x = 462$ mm; $z = 50$ mm and 950 mm).

For strains in horizontal bars, they are given at points where strain gauges located nearest to major diagonal crack in order to observe whether the bars have yielded during testing. Since there is no major diagonal crack in specimen J7 as observed in specimens J5 and J6, thus the points plotted here are taken from the same coordinate as per specimens J5 and J6 in order to make consistent comparison. The plot is presented on Figure 4.84. It should be noted that strain gauge located at coordinate of $z = 600$ mm was not functioning at all during testing and hence there was no data recorded for this particular coordinate. From the figure, it can be seen that all horizontal bars located at upper part of wall yielded during testing while horizontal bars located at middle part of wall suffered high tensile strains and the strain values are very close to

yielding strain. For horizontal bars located at bottom part of wall, there was only one bar that suffered high tensile strains near to its yielding strain while the rest had less tensile strains. In average, the strain values of horizontal bars are much higher than that of vertical bars. This means that for specimen J7, horizontal bars are more effective than vertical bars in contributing to the overall shear strength of the specimen which is also observed in specimens J5 and J6.

Figure 4.84 Strains of horizontal bars at points nearest to major diagonal crack.

Another observation of horizontal bars strains is done for stirrups in the boundary elements. Based on numerical data, the average strain values of stirrups in boundary elements is very low and insignificant. Therefore, the strain values are not plotted here. This means that the

stirrups in the boundary elements do not contribute much to the overall shear strength of the specimen.

4.7 Discussions and Conclusions

In this section, summary and discussions of the experimental results are presented, including effect of height to length ratio, web reinforcement ratios, and presence of boundary elements to the overall behaviour of HSC walls tested, wall shear strengths and deformations. In addition, the important findings from this experiment are summarized as well.

4.7.1 Summary of Experimental Results

The summary of experimental results is given in Table 4.5. Experimental wall shear strength (V_{exp}) is defined as the maximum lateral force recorded (either in positive or negative directions) before first strength degradation observed. This experimental wall shear strength later is used for evaluating building code provisions or analytical model predictions. Furthermore, the experimental wall shear strength is also used to calculate the normalized average shear stress.

In general, it can be concluded that all specimens failed in brittle shear mode. As can be seen in Figure 4.85, in most specimens, the lateral force drops significantly after reaching its peak point. In addition, the drift level at the peak lateral force is mostly below 1.00% for most specimens. From the hysteretic behaviour, it can be concluded that most specimens have low ductility and low energy dissipation capacity. The hysteretic behaviour of HSC walls failing in shear tested in this study is similar to specimens tested by other researchers (Kabeyasawa and Hiraishi 1998). In their experiment, a set of ten HSC low rise walls with concrete compressive strength ranging from 60 to 120 MPa were tested under vertical axial loading and cyclic lateral loading to investigate the shear behaviour. Their specimens failed mostly in web crushing mode at drift level about 0.75% which is also quite similar to specimens tested in this study.

Table 4.5 Summary of experimental results.

No.	Specimen ID	f'_c (MPa)	A_g (mm ²)	H_w/L_w	$P/[f'_c A_g]$	ρ_v	ρ_h	Boundary Elements	V_{max} (kN)	Drift at V_{max} (%)	V_{exp} (kN)	$V_{exp}/[A_w \sqrt{f'_c}]$
1	J1	103.3	196000	1.0	0.05	0.0028	0.0028	YES	+892.29 -1209.60	+0.39 -0.79	1209.60	1.19
2	J2	96.8	196000	1.0	0.05	0.0075	0.0028	YES	+1264.75 -1270.82	+0.80 -0.68	1270.82	1.29
3	J3	110.7	196000	1.0	0.05	0.0028	0.0075	YES	+1402.76 -1458.85	+0.79 -0.76	1458.85	1.39
4	J4	93.5	111200	1.0	0.05	0.0028	0.0028	NO	+810.74 -826.12	+0.54 -0.73	810.74	0.84
5	J5	103.3	196000	2.0	0.05	0.0028	0.0028	YES	+595.76 -556.97	+0.70 -0.70	595.76	0.59
6	J6	96.8	196000	2.0	0.05	0.0075	0.0028	YES	+724.14 -673.00	+0.80 -0.71	724.14	0.74
7	J7	110.7	196000	2.0	0.05	0.0028	0.0075	YES	+894.77 -854.02	+1.17 -0.99	894.77	0.85

Figure 4.85 Envelope curves of specimens tested in this study.

Another thing worth mentioning is that the specimens tested in this study have similar strength in positive and negative directions. The difference of maximum strength in positive and negative directions is less than 10% for all specimens, except for specimen J1 (see Table 4.5) in which the testing could not proceed for positive direction starting from drift level of +0.40% onwards. For example, specimen J4 (see Figure 4.50), it reaches its maximum strength in positive direction at drift level of +0.54% and after that strength degradation is observed in subsequent cycles together with crushing of bottom edge of left column. However, in negative direction of loading, the maximum strength is reached at later stage of loading, i.e. drift level of -0.73% and even the value is slightly higher than maximum strength in positive direction. This means that diagonal cracks occurred in one direction of loading are not affecting the strength in another direction as long as web crushing has not been

observed, and therefore the specimen has similar strength in positive and negative directions.

For wall deformation components, general trend observed for all specimens is that as total wall drift increases, contribution of flexure deformation to total wall drift decreases while it is the opposite for contribution of shear deformation (see Figures 4.27, 4.36, 4.45, 4.54, 4.62, 4.71, 4.80). For specimens with height to length ratio of 1.0, shear deformation is the major contributor of total wall drift at all stages of loading whereas for specimens with height to length ratio of 2.0, flexure deformation is the major contributor of total wall drift at early stage of loading up to point of maximum forces while after that shear deformation becomes the major contributor. In addition, for specimens with boundary elements, sliding shear deformation contribution is very small and negligible while for specimen without boundary elements, sliding shear deformation starts to give significant contribution after the specimen reaches its maximum forces in positive and negative directions. This finding is in agreement with experiment conducted by Corley et al. (1981). They concluded that boundary elements or flanges could prevent sliding shear failure by providing stiff dowels.

In general, strain distributions in web reinforcement are highly non-linear for all specimens. Furthermore, in most specimens, average strain values for horizontal web reinforcement are higher than those of vertical web reinforcement (see Figures 4.29, 4.31, 4.47, 4.49, 4.56, 4.57, 4.64, 4.66, 4.73, 4.75, 4.82, 4.84). It means that for walls having height to length ratio of 1.0 and 2.0, horizontal web reinforcement is more effective than vertical web reinforcement in contributing to overall shear strength. Previous experiment by Barda et al. (1977) which investigated low rise walls having height to length ratio of 0.25, 0.5, and 1.0 concluded that for walls having height to length ratio of 0.25 and 0.5, the average strain values of vertical web reinforcement are higher than those of horizontal web reinforcement while for walls having height to length ratio of 1.0, the average strain values are approximately equal between

vertical and horizontal web reinforcement. Therefore, based on experiment results by Barda et al. (1977) and this study, it can be concluded that vertical web reinforcement is more effective than horizontal web reinforcement for walls having height to length ratio less than 1.0 while the opposite is true for walls having height to length ratio more than 1.0.

For vertical reinforcement in boundary elements, different trend of strain values is observed between specimens tested. Thus, it is difficult to make any conclusions based on the results obtained.

For stirrups (horizontal reinforcement) in boundary elements, it can be concluded that they do not contribute much to the overall shear strength of wall since the strain values are very small and insignificant in all specimens tested in this study.

4.7.2 Effect of Height to Length Ratio

Effect of height to length ratio to overall behaviour of RC wall is investigated by comparing similar specimens that have the same properties, except the height to length ratio, i.e. specimen J1 to J5, specimen J2 to J6, and specimen J3 to J7. The effect of height to length ratio will be discussed in relation to wall shear strength, drift level or ductility, hysteretic behaviour, and wall deformation components.

In relation to wall shear strength, the effect of height to length ratio is discussed using normalized average shear stress rather than experimental wall shear strength. This is done to take into account the difference of concrete compressive strength between wall specimens. Furthermore, in order to obtain the complete picture, envelope curves of the above mentioned specimens are compared and presented on Figures 4.86 to 4.88.

Figure 4.86 Comparison of envelope curves of specimens J1 and J5.

From the figures as well as data provided in Table 4.5, it is clear that for RC walls failing in shear mode, walls having lower height to length ratio can exhibit greater shear strength. The normalized average shear stresses for walls having height to length ratio of 1.0 are between 1.6-2.0 times of those for walls having height to length ratio of 2.0. This may be due to occurrence of cracks causes more severe stiffness degradation for higher specimens. With similar drift level at peak lateral forces between lower and higher specimens failing in shear mode, except for specimen J7, higher specimens with more severe stiffness degradation will have lower shear strength as compared to lower specimens. This finding is in agreement with experiment results conducted by Barda et al. (1977) which also investigated the effect of height to length ratio to overall shear strength of RC wall. In their experiment, it has been found that increasing height to length ratio from 0.5 to 1.0 reduces wall shear strength by 20% while in this study, it has been found that increasing height to length ratio

from 1.0 to 2.0 reduces wall shear strength about 40%-50% (see Table 4.5). Hence, it can be concluded that every increment of 0.5 in wall height to length ratio (ranging from height to length ratio of 0.5 to 2.0), the shear strength will be reduced by 20%.

Figure 4.87 Comparison of envelope curves of specimens J2 and J6.

The effect of height to length ratio to drift level at peak lateral forces does not seem to be significant for RC walls failing in shear. Most of specimens, except for specimen J7, have drift level at peak lateral forces below 1.00% (see Table 4.5) which is typical value for brittle shear failure. For specimen J7, the drift level at peak lateral force can reach up to 1.17% due to combination factor of higher height to length ratio as well as higher horizontal web reinforcement ratio. Nevertheless, walls having higher height to length ratio seem to have slightly more ductility than walls having lower height to length ratio. It is shown by the lateral force that does not drop significantly in subsequent cycle after reaching

its peak point for walls with higher height to length ratio whereas it is not the case for walls with lower height to length ratio (see Figures 4.86 to 4.88).

Figure 4.88 Comparison of envelope curves of specimens J3 and J7.

From the hysteretic behaviour, it can be seen that for walls having higher height to length ratio, pinching behaviour still can be observed after strength degradation occurred (see Figures 4.60, 4.69, 4.78). On the other hand, for walls with lower height to length ratio, no pinching behaviour is observed and the behaviour is similar to concrete failing in compression (see Figures 4.25, 4.34, 4.43).

For wall deformation components, it can be seen that for walls having higher height to length ratio, although failing in shear, at early stage of loading, flexure deformation still dominates up to the point of maximum forces (see Figures 4.62, 4.71, 4.80) while for walls with lower height to length ratio, shear deformation dominates at all stages of loading (see

Figures 4.27, 4.36, 4.45). This means that shear behaviour is more pronounced for walls with lower height to length ratio.

4.7.3 Effect of Web Reinforcement Ratio

Effect of web reinforcement ratio to overall behaviour of RC wall is investigated by comparing similar specimens that have the same properties, except the web reinforcement ratio, i.e. specimen J1 to J2 and J3, specimen J5 to J6 and J7. The effect of web reinforcement ratio will be discussed in relation to wall shear strength and drift level or ductility.

In relation to wall shear strength, the effect of web reinforcement ratio is also discussed using normalized average shear stress rather than experimental wall shear strength. This is done to take into account the difference of concrete compressive strength between wall specimens. In addition, it will be discussed separately between walls with height to length ratio of 1.0 and 2.0 since the effect of web reinforcement ratio may not be the same. Furthermore, in order to obtain the complete picture, envelope curves of the above mentioned specimens are compared and presented on Figures 4.89 and 4.90.

For walls with lower height to length ratio (1.0), it can be seen from the experiment results that both vertical and horizontal web reinforcement have contribution to increase wall shear strength although the increment is not so significant (see Table 4.5). Increasing web reinforcement ratio from 0.28% to 0.75% resulted in 8.40% and 16.81% increment in normalized average shear stress for vertical and horizontal web reinforcement, respectively. It means that horizontal web reinforcement is more effective than vertical web reinforcement in contributing to wall shear strength for walls with height to length ratio of 1.0. This finding is in agreement with experiment results conducted by Gupta and Rangan (1998) and Farvashany et al. (2008) which investigated the effect of web reinforcement ratio to shear strength of HSC low rise walls with similar height to length ratio. In experiment by Gupta and Rangan (1998), it is found that increasing vertical web reinforcement ratio from 1.00% to

1.50% only resulted in 3.36% increment of normalized average shear stress while increasing horizontal web reinforcement ratio from 0.50% to 1.00% resulted in 5.88% increment of normalized average shear stress. In experiment by Farvashany et al. (2008), actually they concluded that vertical web reinforcement is more effective than horizontal web reinforcement in contributing to shear strength of RC wall. However, based on their data, it is shown that increasing vertical web reinforcement ratio from 0.75% to 1.26% resulted in 10.00% increment of normalized average shear stress whereas increasing horizontal web reinforcement ratio from 0.47% to 0.75% resulted in 8.40% increment of normalized average shear stress. It is worth noting that the amount of increment of horizontal web reinforcement ratio is less than vertical web reinforcement ratio. Therefore, if horizontal web reinforcement ratio is increased by the same amount as that of vertical web reinforcement, the increment of normalized average shear stress may be higher than 10.00%. Hence, it still can be concluded that horizontal web reinforcement is more effective than vertical web reinforcement in contributing to shear strength for walls with height to length ratio of 1.25.

For walls with higher height to length ratio (2.0), it can be seen from experiment results that both vertical and horizontal web reinforcement have contribution to increase wall shear strength and the effect is more significant here than that in walls with lower height to length ratio (see Table 4.5). Increasing web reinforcement ratio from 0.28% to 0.75% resulted in 25.42% and 44.07% increment in normalized average shear stress for vertical and horizontal web reinforcement, respectively. This means that horizontal web reinforcement is more effective than vertical web reinforcement in contributing to wall shear strength for walls with height to length ratio of 2.0.

Figure 4.89 Comparison of envelope curves of specimens J1, J2, and J3.

Previous experiment by Barda et al. (1977) which also investigated effect of web reinforcement ratio to RC wall shear strength concluded that for walls having height to length ratio of 0.5 or less, horizontal web reinforcement has very little contribution to the shear strength while vertical web reinforcement is effective as shear reinforcement. However, contribution of vertical web reinforcement to shear strength reduces as the height to length ratio increases. Therefore, from results of previous experiments as well as experiment in this study, it can be concluded that vertical web reinforcement is more effective than horizontal web reinforcement in contributing to overall shear strength for walls having height to length ratio less than 1.0 whereas the opposite is true for walls having height to length ratio equal or more than 1.0.

Figure 4.90 Comparison of envelope curves of specimens J5, J6, and J7.

Another thing worth mentioning is that the contribution of web reinforcement to overall shear strength of RC wall is more significant for walls with higher height to length ratio (see Table 4.5). Increasing vertical web reinforcement ratio from 0.28% to 0.75% resulted in increment of normalized average shear stress by 0.10 and 0.15 for walls with lower and higher height to length ratio, respectively while increasing horizontal web reinforcement ratio from 0.28% to 0.75% resulted in increment of normalized average shear stress by 0.20 and 0.26 for walls with lower and higher height to length ratio, respectively. If these increments are converted into percentage of normalized average shear stress as presented previously, then it can be concluded that contribution of web reinforcement to overall shear strength of RC wall is more significant for walls with higher height to length ratio whereas for walls with lower height to length ratio, concrete contribution is very dominant. In addition, from these results, it can be concluded that concrete

contribution to overall shear strength of RC wall decreases as height to length ratio increases while contribution of web reinforcement increases as height to length ratio increases.

For drift level at peak lateral forces, it can be seen that increasing web reinforcement ratio does not seem to have significant effect for walls with lower height to length ratio (see Figure 4.89). This is because for walls with lower height to length ratio, the hysteretic behaviour looks similar to concrete failing in compression and the failure mode is governed by web crushing. Hence, increasing web reinforcement does not have much effect on drift level at peak lateral forces. For walls with higher height to length ratio (see Figure 4.90), increasing horizontal web reinforcement ratio can increase drift level at peak lateral force to become 1.17%. This may be due to more horizontal web reinforcement can help to limit diagonal crack width and hence delaying strength degradation to later stage of loading. This can be seen from comparing crack propagation on specimens J5 and J6 to specimen J7. Specimens J5 and J6 with lesser horizontal web reinforcement have one major diagonal crack that opens quite wide as seen in Figures 4.59 and 4.68 whereas in specimen J7 there is no single major diagonal crack that opens quite wide as observed in Figure 4.77. Therefore, for specimen J7, strength degradation can be delayed to later stage of loading until web crushing occurred.

4.7.4 Effect of Boundary Elements

Effect of boundary elements to overall behaviour of RC wall is investigated by comparing similar specimens that have the same properties, except the presence of boundary elements, i.e. specimen J1 to J4. The effect of boundary elements will be discussed in relation to wall shear strength and wall deformation components.

In relation to wall shear strength, the effect of boundary elements is also discussed using normalized average shear stress rather than experimental wall shear strength. This is done to take into account the difference of concrete compressive strength between wall specimens.

Furthermore, in order to obtain the complete picture, envelope curves of the above mentioned specimens are compared and presented on Figure 4.91.

Figure 4.91 Comparison of envelope curves of specimens J1 and J4.

From the experiment results, it can be seen that the presence of large boundary elements can increase significantly the shear strength of RC wall (see also Table 4.5). The normalized average shear stress is increased by 0.35 or 41.67% for specimen J1 as compared to J4. Since failure in both specimens is associated with crushing of compression zone at the bottom edge of wall, specimen J1 with larger compression zone area fails at significantly higher force than specimen J4. This finding is in agreement with experiment conducted by Corley et al. (1981).

For wall deformation components, it is worth noting that for specimen J4 (see Figure 4.54), after reaching the point of maximum forces, sliding shear deformation starts to give significant contribution to total wall drift

while this phenomenon is not observed in specimen J1 with boundary elements. For specimen J1 (see Figure 4.27), sliding shear deformation is small and not significant at all stages of loading. This finding is also in agreement with experiment conducted by Corley et al. (1981) which concluded that boundary element or flange could help to prevent sliding shear failure by providing stiff dowels.

4.7.5 Conclusions

Findings from this experiment study are summarized as follows:

- All specimens failed in brittle shear mode and have low ductility and low energy dissipation capacity. Drift level at peak lateral forces is below 1.00% for most specimens, except for specimen J7 which is due to combination factor of higher height to length ratio and higher horizontal web reinforcement ratio.
- All specimens have similar shear strength in positive and negative directions. It means that diagonal cracks occurred in one direction of loading are not affecting the strength in another direction as long as web crushing has not been observed.
- For wall deformation components, general trend observed is that as total wall drift increases, contribution of flexure deformation to total wall drift decreases while it is the opposite for contribution of shear deformation. For walls with higher height to length ratio, flexure deformation still dominates up to point of maximum forces whereas for walls with lower height to length ratio, shear deformation dominates at all stages of loading.
- Stirrups in boundary elements do not contribute much to the overall shear strength of RC wall.
- Walls with lower height to length ratio have higher shear strength as compared to the ones with higher height to length ratio. From previous experiment by Barda et al. (1977) and this experiment, it can be concluded that every increment of 0.5 in height to length ratio (for

walls with height to length ratio ranging from 0.5 to 2.0), the overall shear strength will be reduced by 20%.

- For RC wall failing in shear, walls with higher height to length ratio have slightly more ductility than the ones with lower height to length ratio.
- For walls with height to length ratio of 1.0 and 2.0, horizontal web reinforcement is more effective than vertical web reinforcement in contributing to shear strength of RC wall. However, for walls with height to length ratio less than 1.0, previous experiment by Barda et al. (1977) concluded that vertical web reinforcement is more effective than horizontal web reinforcement in contributing to shear strength of RC wall. Therefore, it can be concluded that for walls with height to length ratio less than 1.0, vertical web reinforcement is more effective whereas for ones with height to length ratio equal or more than 1.0, horizontal web reinforcement is more effective.
- Contribution of web reinforcement to overall shear strength of RC wall is more significant for walls with higher height to length ratio whereas for ones with lower height to length ratio, concrete contribution is very dominant. In addition, concrete contribution to overall shear strength of RC wall decreases as height to length ratio increases while contribution of web reinforcement increases as height to length ratio increases.
- Increasing amount of web reinforcement does not have significant effect to drift level at peak lateral forces for walls with lower height to length ratio while for ones with higher height to length ratio, increasing horizontal web reinforcement can increase drift level at peak lateral force to exceed 1.00%.
- The presence of large boundary elements can increase significantly the shear strength of RC wall. Furthermore, it also can reduce sliding shear deformation to negligible level. This finding is in agreement with experiment conducted by Corley et al. (1981).

CHAPTER 5

ANALYTICAL MODELS FOR PREDICTING RC WALL SHEAR STRENGTH

5.1 Overview

In this chapter, the author will describe his newly proposed analytical models that are based on truss analogy principles for use in calculating the shear strengths of RC walls. The strength prediction results of past experiments on RC walls as well as those from this study using the proposed models are presented and compared with predictions from building codes and other analytical models. Furthermore, examples of RC wall shear strength calculation using the proposed models are given as well. Lastly, conclusions regarding the proposed models are summarized.

5.2 Truss Model for Predicting RC Wall Shear Strength

As one of the objectives of this study is to develop an analytical model for predicting RC wall shear strengths, in this section, an analytical model that has been developed based on softened truss model (Hsu 1988) is described as follows.

5.2.1 Equilibrium Conditions

Consider a typical RC wall panel as displayed in Figure 5.1a. The applied external shear force (V) and applied external axial force (P) are assumed to be distributed uniformly throughout the wall panel by means of a rigid top beam or slab. As the external shear force increases in magnitude, diagonal cracks will occur in the wall panel which are illustrated by dashed lines in Figure 5.1a, forming a series of concrete diagonal struts with certain angle (θ) to the horizontal axis. State of stresses in the wall panel are illustrated in Figure 5.1b, that the wall panel can be presented as combination of diagonal concrete struts and vertical and horizontal web reinforcements. In this model, the principal stress directions of the concrete are assumed to coincide with the direction of

cracks and the web reinforcement is assumed to take only axial stresses, neglecting any dowel action of web reinforcement.

(a) Typical RC wall and its component forces and web reinforcement.

(c) Wedge A-o-p. Area of side op is taken to be one-unit area. Therefore, $A_o = \cos \theta$ and $A_p = \sin \theta$. Forces from the distributed web reinforcement are represented by $\rho_v f_v$ and $\rho_h f_h$.

(b) r - d are assumed to be the principal directions. So, τ_{rd} and τ_{dr} become equal to zero.

(d) Wedge B-m-n. Area of side mn is taken to be one-unit area. Therefore, $B_m = \cos \theta$ and $B_n = \sin \theta$. Forces from the distributed web reinforcement are represented by $\rho_v f_v$ and $\rho_h f_h$.

Figure 5.1 State of stresses in a typical RC wall panel.

Since the constitutive laws of concrete are presented in terms of principal stresses in the r-d coordinate system, thus it is necessary to perform stress transformations of concrete from r-d coordinate system into v-h coordinate system (the vertical and horizontal axes of the wall panel). The procedure of stress transformations is explained as follows. Consider triangular wedge A-o-p of the RC wall panel as displayed in Figure 5.1c. Let the thickness of the RC wall panel be one unit and the length of the side op be one unit as well. Hence, the area of the op side is one-unit area and the areas of Ao and Ap sides are $(\cos\theta)$ and $(\sin\theta)$, respectively. In Figure 5.1c, the vertical force equilibrium can be presented by equation (5.1) which can be rearranged as equation (5.2).

$$\sigma_v - \sigma_d \sin^2\theta - \sigma_r \cos^2\theta - \rho_v f_v = 0 \quad (5.1)$$

$$\sigma_v = \sigma_d \sin^2\theta + \sigma_r \cos^2\theta + \rho_v f_v \quad (5.2)$$

Similarly, in Figure 5.1d, the horizontal force equilibrium can be presented by equation (5.3) which can be rearranged as equation (5.4).

$$\sigma_h - \sigma_d \cos^2\theta - \sigma_r \sin^2\theta - \rho_h f_h = 0 \quad (5.3)$$

$$\sigma_h = \sigma_d \cos^2\theta + \sigma_r \sin^2\theta + \rho_h f_h \quad (5.4)$$

In order to obtain shear stress in the RC wall panel (τ_{vh}), either the horizontal force equilibrium in Figure 5.1c or the vertical force equilibrium in Figure 5.1d can be used. The horizontal force equilibrium in Figure 5.1c can be presented by equation (5.5) which can be rearranged as equation (5.6). Hence, the transformed average stress equilibriums in the RC wall panel are represented by equations (5.2), (5.4), and (5.6).

$$\tau_{vh} + \sigma_d \sin\theta \cos\theta - \sigma_r \cos\theta \sin\theta = 0 \quad (5.5)$$

$$\tau_{vh} = (\sigma_r - \sigma_d) \sin\theta \cos\theta \quad (5.6)$$

where:

- σ_v = applied normal stress in vertical axis, positive for tension
- σ_h = applied normal stress in horizontal axis, positive for tension
- τ_{vh} = average shear stress in v-h coordinate system and is due to shear force acting on the wall
- σ_d = principal stress of concrete in d-axis, positive for tension
- σ_r = principal stress of concrete in r-axis, positive for tension
- θ = angle of diagonal concrete strut (d-axis) with respect to horizontal axis
- ρ_v = average vertical web reinforcement ratio
- ρ_h = average horizontal web reinforcement ratio
- f_v = average stress in the vertical web reinforcement
- f_h = average stress in the horizontal web reinforcement

5.2.2 Compatibility Conditions

Similar as in the case of equilibrium conditions, for compatibility conditions, it is necessary to perform strain transformations from r-d coordinate system into v-h coordinate system since the constitutive laws of concrete is presented in terms of principal strains in the r-d coordinate system. Transformation equations for strains are similar to transformation equations for stresses (Gere 2004) with σ_v corresponds to ε_v , σ_h corresponds to ε_h , and τ_{vh} corresponds to $\gamma_{vh}/2$. Thus, the compatibility equations of the RC wall panel can be presented by equations (5.7) to (5.9).

$$\varepsilon_v = \varepsilon_d \sin^2 \theta + \varepsilon_r \cos^2 \theta \quad (5.7)$$

$$\varepsilon_h = \varepsilon_d \cos^2 \theta + \varepsilon_r \sin^2 \theta \quad (5.8)$$

$$\gamma_{vh} = 2(\varepsilon_r - \varepsilon_d) \sin \theta \cos \theta \quad (5.9)$$

where:

- ε_v = normal strain in vertical axis, positive for tension

- ε_h = normal strain in horizontal axis, positive for tension
 γ_{vh} = shear strain in v-h coordinate system
 ε_d = principal strain in d-axis, positive for tension
 ε_r = principal strain in r-axis, positive for tension
 θ = angle of diagonal concrete strut (d-axis) with respect to horizontal axis

5.2.3 Constitutive Law of Materials

In this section, constitutive laws of steel bars and reinforced concrete are described with supporting equations and graphs. For steel bars, the stress-strain relationship is assumed to be elastic-perfectly plastic with maximum average stress is set to be the lesser value of $0.8f_y$ or 500 MPa as presented in Figure 5.2 and equations (5.10) to (5.11). It has been found in this experimental study that not all (100%) web reinforcements yielded during testing. Thus, it is assumed that the maximum average stresses are about 80% of their yield strengths. Moreover, it is also assumed that the maximum stresses are limited to 500 MPa for typical shear reinforcement.

$$f_v = E_s \varepsilon_v ; \text{ for } 0 < \varepsilon_v < \min(0.8\varepsilon_{yv}; 0.0025) \quad (5.10a)$$

$$f_v = \min(0.8f_{yv}; 500\text{MPa}) ; \text{ for } \varepsilon_v \geq \min(0.8\varepsilon_{yv}; 0.0025) \quad (5.10b)$$

$$f_h = E_s \varepsilon_h ; \text{ for } 0 < \varepsilon_h < \min(0.8\varepsilon_{yh}; 0.0025) \quad (5.11a)$$

$$f_h = \min(0.8f_{yh}; 500\text{MPa}) ; \text{ for } \varepsilon_h \geq \min(0.8\varepsilon_{yh}; 0.0025) \quad (5.11b)$$

where:

- f_v = average stress in the vertical web reinforcement (MPa)
 f_h = average stress in the horizontal web reinforcement (MPa)
 f_{yv} = yield strength of vertical web reinforcement (MPa)
 f_{yh} = yield strength of horizontal web reinforcement (MPa)
 E_s = modulus elasticity of steel bars (200000 MPa)
 ε_v = average strain in the vertical web reinforcement
 ε_h = average strain in the horizontal web reinforcement

ε_{yv} = yield strain of vertical web reinforcement

ε_{yh} = yield strain of horizontal web reinforcement

Figure 5.2 Average stress-strain curve of steel bars.

For stress-strain curve of concrete in tension, it is taken as suggested by Belarbi and Hsu (1994) and Pang and Hsu (1995) as shown in Figure 5.3 with some modifications after the concrete has cracked. Before the concrete cracks, the stress-strain curve is assumed to be linear as presented by equation (5.12a). After the concrete has cracked, instead of using equation as suggested by Belarbi and Hsu (1994) and Pang and Hsu (1995), in this model the residual tensile stress in concrete is simply assumed to be $0.02f'_c$. This assumption is based on experimental data on stress-strain curve of concrete in tension (Reinhardt et al. 1986; Yankelevsky and Reinhardt 1989; Laskar et al. 2007) as shown in Figure 5.4a to 5.4c. As can be seen in those figures, the residual tensile stress of concrete is about 20 percent of its tensile strength. Hence, assuming its tensile strength is about 10 percent of its compressive strength, the residual tensile stress of concrete can be assumed to be 2 percent of its compressive strength.

Figure 5.3 Average stress-strain curve of concrete in tension.

Figure 5.4a Stress-strain curve of concrete in tension under monotonic (left) and cyclic (right) loadings (Reinhardt et al. 1986).

Figure 5.4b Stress-strain curve of concrete in tension under monotonic (type I and II) and cyclic (type III and IV) loadings (Yankelevsky and Reinhardt 1989).

Figure 5.4c Average stress-strain curve of concrete in tension (Laskar et al. 2007).

$$\sigma_r = E_c \varepsilon_r ; \text{ for } \varepsilon_r \leq 0.00008 \quad (5.12a)$$

$$\sigma_r = 0.02f'_c ; \text{ for } \varepsilon_r > 0.00008 \quad (5.12b)$$

where:

σ_r = principal stress of concrete in r-axis, positive for tension (MPa)

E_c = modulus of elasticity of concrete which is taken as $(3320\sqrt{f'_c} + 6900)$ MPa according to ACI 363 (1992)

ε_r = principal strain of concrete in r-axis, positive for tension

f_{cr} = cracking stress of concrete (MPa)

f'_c = concrete cylinder compressive strength (MPa)

In this model, the stress-strain curve of concrete in compression is taken as defined by Hsu and his colleagues (Belarbi and Hsu 1994; Belarbi and Hsu 1995; Pang and Hsu 1995; Zhang and Hsu 1998) which is shown in equation (5.13) and Figure 5.5. This relationship has already taken into account the softening behaviour of cracked concrete because of tensile strain in perpendicular direction.

Figure 5.5 Average stress-strain curve of concrete in compression.

$$\sigma_d = -\zeta f'_c \left[2 \left(\frac{\varepsilon_d}{-\zeta \varepsilon_o} \right) - \left(\frac{\varepsilon_d}{-\zeta \varepsilon_o} \right)^2 \right] ; \text{ for } -\varepsilon_d \leq \zeta \varepsilon_o \quad (5.13a)$$

$$\sigma_d = -\zeta f'_c \left[1 - \left(\frac{\frac{\varepsilon_d}{-\zeta \varepsilon_o} - 1}{\frac{2}{\zeta} - 1} \right)^2 \right] ; \text{ for } -\varepsilon_d > \zeta \varepsilon_o \quad (5.13b)$$

$$\zeta = \left(\frac{5.8}{\sqrt{f'_c}} \leq 0.9 \right) \left(\frac{1}{\sqrt{1+400\varepsilon_r}} \right) \quad (5.14)$$

where:

σ_d = principal stress of concrete in d-axis, positive for tension (MPa)

ζ = softening factor of concrete

f'_c = concrete cylinder compressive strength (in MPa)

ε_d = principal strain of concrete in d-axis, positive for tension

ε_r = principal strain of concrete in r-axis, positive for tension

ε_o = strain of concrete at maximum compressive stress which is taken as $[0.002 + 0.001 \times (f'_c - 20) / 80]$ according to Foster and Gilbert (1996)

5.2.4 Solution Procedures

Based on equilibrium conditions, compatibility conditions, and constitutive law of materials, there are thirteen variables in total (σ_v , σ_h , σ_d , σ_r , f_v , f_h , ε_v , ε_h , ε_d , ε_r , τ_{vh} , γ_{vh} , θ) with ten equations available (5.2, 5.4, 5.6, 5.7 to 5.13). Thus, it is necessary to determine three known variables in order to obtain the solutions. Firstly, known terms such as σ_v and σ_h can be replaced by their respective values. σ_v is the applied normal stress in vertical axis caused by external axial force (P/A_g) and the value is positive for tension and negative for compression. The value of σ_h is mostly zero in typical RC wall. Lastly, since this model aims to obtain the nominal shear strength of RC walls, the value of ε_d can be determined as $-\zeta \varepsilon_o$ as it relates to maximum softened compressive stress in the diagonal concrete strut as presented on Figure 5.5.

After determining three known variables, the next step is to assume a certain value of ε_r . Hence, the value of σ_r and σ_d can be calculated using

equations (5.12) and (5.13), respectively. After that, the values of ε_v and ε_h are calculated with the procedure described below. By rearranging equation (5.2), the value of $(\sin^2 \theta)$ can be expressed as equation (5.15). Substituting equation (5.15) into equation (5.7), then the value of ε_v can be expressed as equation (5.16a). In this case, the value of f_v can be substituted by equation (5.10a) assuming that the reinforcement has not reached its maximum strength, i.e. $\min(0.8f_{yv}; 500\text{MPa})$. Thus, equation (5.16a) can be rearranged as equation (5.16b) in order to calculate the value of ε_v . If the value of ε_v calculated using equation (5.16b) is greater than the lesser value of $0.8\varepsilon_{yv}$ and 0.0025 , then the value of ε_v has to be recalculated using equation (5.16a) by putting the value of f_v as equation (5.10b).

$$\sin^2 \theta = \frac{-\sigma_v + \sigma_r + \rho_v f_v}{-\sigma_d + \sigma_r} \quad (5.15)$$

$$\varepsilon_v = \varepsilon_r + \left(\frac{-\sigma_v + \sigma_r + \rho_v f_v}{-\sigma_d + \sigma_r} \right) (\varepsilon_d - \varepsilon_r) \quad (5.16a)$$

$$\varepsilon_v = \frac{\varepsilon_r(-\sigma_d + \sigma_v) + \varepsilon_d(-\sigma_v + \sigma_r)}{(-\sigma_d + \sigma_r) - \rho_v E_s(\varepsilon_d - \varepsilon_r)} \quad (5.16b)$$

Subsequently, by rearranging equations (5.7) and (5.8) into equation (5.17), the value of ε_h can be calculated. By doing this, it means that the compatibility conditions have been satisfied. After the values of ε_v and ε_h are calculated, the values of f_v and f_h can be calculated using equations (5.10) and (5.11), respectively. Lastly, to satisfy the equilibrium conditions, equations (5.2) and (5.4) are combined to become equation (5.18). If the total value of left hand side is the same as the total value of right hand side, it means that the equilibrium conditions have been satisfied and the assumed value of ε_r is the correct one. Otherwise, the value of ε_r has to be assumed again to other values and the procedure is repeated until the correct value of ε_r is obtained. After that, the values of θ , τ_{vh} , and γ_{vh} can be calculated using equations (5.15), (5.6), and (5.9), respectively.

$$\varepsilon_v + \varepsilon_h = \varepsilon_d + \varepsilon_r \quad (5.17)$$

$$\sigma_v + \sigma_h = \sigma_d + \sigma_r + \rho_v f_v + \rho_h f_h \quad (5.18)$$

5.2.5 Inclusion of Dowel Action

Nominal shear strength of RC wall (V_n) according to this model consists of contribution from RC wall web panel that has been explained in previous subsections and contribution of dowel action of reinforced boundary elements. The contribution from RC wall web panel can be calculated by multiplying average shear stress (τ_{vh}) as calculated using equation (5.6) by wall web area (A_w). In this model, wall web area (A_w) is defined as wall web thickness (t_w) multiplied by effective depth of wall (d_w). Effective depth of wall can be taken as the distance between center to center of boundary elements or $0.8L_w$ (80% wall length) in case of wall without boundary elements.

The inclusion of dowel action is in agreement with experimental findings by Corley et al. (1981) and is also confirmed by the author's own experimental results. The boundary elements with concentrated reinforcements at both ends of walls can increase the shear strengths of RC walls. In this model, dowel action formula as developed by Baumann and Rusch (1970) is adopted and shown in equation (5.19). This equation was also used by He and Teng (1998) to include dowel action when predicting shear strengths of RC beams. In this equation, the total area of vertical reinforcement in one boundary element (A_{sb}) is converted to single dowel bar with same area and has equivalent bar diameter (d_{be}) which is larger than diameter of individual vertical bar in the boundary element. Then, effective width of the boundary element (b_{ef}) can be calculated by subtracting the overall width of the boundary element (b_f) with the equivalent bar diameter (d_{be}). Here, as shown in Figure 5.6, the overall width of boundary element (b_f) does not need to be taken greater than half of wall height plus wall web thickness ($0.5H_w + t_w$) as suggested by ACI 318 Chapter 18 (2014). The dowel force (D_u) is then added as an

additional component of the shear strength of RC wall. Thus, nominal shear strength of RC wall (V_n) according to this model can be expressed by equation (5.20). Note that in this model, the dowel force is considered for one boundary element only (that is in tension) since the dowel force will become active because of crack opening.

$$D_u = 1.64b_{ef}d_{be}\sqrt[3]{f'_c} \quad (5.19)$$

$$V_n = (\sigma_r - \sigma_d) \sin \theta \cos \theta t_w d_w + 1.64b_{ef}d_{be}\sqrt[3]{f'_c} \quad (5.20)$$

where:

D_u = dowel force of vertical reinforcement in one boundary element (Newton)

b_{ef} = effective width of boundary element (mm)

d_{be} = equivalent bar diameter (mm)

f'_c = concrete cylinder compressive strength (MPa)

V_n = nominal shear strength of RC wall (Newton)

σ_d = principal stress of concrete in d-axis, positive for tension (MPa)

σ_r = principal stress of concrete in r-axis, positive for tension (MPa)

θ = angle of diagonal concrete strut (d-axis) with respect to horizontal axis

t_w = thickness of wall web (mm)

d_w = effective depth of wall (mm)

Figure 5.6 Boundary element considered for dowel action.

5.2.6 Summary

Overall, step by step procedure of this model which is similar to that of softened truss model (Hsu 1988), is summarized as follows:

- 1) Determine the values of σ_v (applied normal stress in vertical axis, which is equal to P/A_g , the value is positive for tension) and σ_h (applied normal stress in horizontal axis, which is equal to zero in most RC walls).
- 2) At ultimate stage, the value of ϵ_d (principal compressive strain of concrete, the value is negative for compression) can be determined as $-\zeta\epsilon_o$ which relates to maximum softened compressive stress in the diagonal concrete strut.
- 3) Assume certain value of ϵ_r (principal tensile strain of concrete, the value is positive for tension).
- 4) Calculate σ_r and σ_d using equations (5.12) and (5.13), respectively.
- 5) Calculate ϵ_v and ϵ_h using equations (5.16) and (5.17), respectively.
- 6) Calculate f_v and f_h using equations (5.10) and (5.11), respectively.
- 7) Check the equilibrium conditions using equation (5.18). If it is satisfied, then go to next step. Otherwise, assume other values of ϵ_r and repeat step 4 to 6 until the equilibrium conditions is satisfied.
- 8) Calculate θ using equation (5.15).
- 9) Calculate D_u using equation (5.19).
- 10) Calculate V_n using equation (5.20).

5.2.7 The Calculation of Shear Strength of RC Wall using the Truss Model

In this section, an example of RC wall shear strength calculation using the truss model is given. As an example, a specimen from the ones tested in this study is used, i.e. specimen J5. The procedure is given as follows:

Specimen J5 data:

Concrete cylinder compressive strength, $f'_c = 103.3 \text{ MPa}$

Wall gross cross section area, $A_g = 196000 \text{ mm}^2$

Axial load applied at wall, $P = 1012 \text{ kN}$ (compression)

Wall height, $H_w = 2000 \text{ mm}$

Wall length, $L_w = 1000 \text{ mm}$

Thickness of wall web, $t_w = 100 \text{ mm}$

Width of boundary element, $b_f = 500 \text{ mm}$

Thickness of boundary element, $t_f = 120 \text{ mm}$

Ratio of vertical reinforcement in boundary element, $\rho_b = 0.0388$

Yield strength of vertical reinforcement in boundary element, $f_{yb} = 630 \text{ MPa}$

Ratio of vertical web reinforcement, $\rho_v = 0.0028$

Yield strength of vertical web reinforcement, $f_{yv} = 610 \text{ MPa}$

Ratio of horizontal web reinforcement, $\rho_h = 0.0028$

Yield strength of horizontal web reinforcement, $f_{yh} = 610 \text{ MPa}$

Experimental wall shear strength, $V_{exp} = 595.76 \text{ kN}$

Calculation of nominal shear strength (V_n) using the truss model:

1. Determine the value of σ_v and σ_h .

$$\sigma_v = -P/A_g = -1012000 / 196000 = -5.163 \text{ MPa}$$

$$\sigma_h = 0$$

2. Determine the value of ε_d .

$$\varepsilon_d = -\zeta \varepsilon_o$$

$$\varepsilon_o = 0.002 + 0.001 * (f'_c - 20) / 80$$

$$\varepsilon_o = 0.002 + 0.001 * (103.3 - 20) / 80$$

$$\varepsilon_o = 0.003041$$

$$\zeta = \left(\frac{5.8}{\sqrt{f'_c}} \leq 0.9 \right) \left(\frac{1}{\sqrt{1 + 400 \varepsilon_r}} \right)$$

$$\zeta = \left(\frac{5.8}{\sqrt{103.3}} \leq 0.9 \right) \left(\frac{1}{\sqrt{1 + 400 \times \varepsilon_r}} \right)$$

Since ε_r has not been defined yet, thus ε_d is still a function of ε_r .

3. Assume a certain value of ε_r .

The value of ε_r should be assumed first in order the analysis can be proceed. Since it requires many iterations, in this example, only the value of ε_r that satisfies equilibrium is presented here.

Let the value of $\varepsilon_r = 0.085008$ (the value is obtained after several iterations).

Then:

$$\zeta = \left(\frac{5.8}{\sqrt{f'_c}} \leq 0.9 \right) \left(\frac{1}{\sqrt{1 + 400\varepsilon_r}} \right)$$

$$\zeta = \left(\frac{5.8}{\sqrt{103.3}} \leq 0.9 \right) \left(\frac{1}{\sqrt{1 + 400 \times 0.085008}} \right)$$

$$\zeta = 0.0965$$

$$\varepsilon_d = -\zeta\varepsilon_o$$

$$\varepsilon_d = -0.0965 \times 0.003041$$

$$\varepsilon_d = -0.000293$$

4. Calculate σ_r and σ_d using equations (5.12) and (5.13), respectively.

$$\sigma_r = 0.02f'_c$$

$$\sigma_r = 0.02 \times 103.3$$

$$\sigma_r = 2.066 \text{ MPa}$$

$$\sigma_d = -\zeta f'_c \left[2 \left(\frac{\varepsilon_d}{-\zeta\varepsilon_o} \right) - \left(\frac{\varepsilon_d}{-\zeta\varepsilon_o} \right)^2 \right]$$

$$\sigma_d = -0.0965 \times 103.3 [2(1) - (1)^2]$$

$$\sigma_d = -9.968 \text{ MPa}$$

5. Calculate ε_v and ε_h using equations (5.16) and (5.17), respectively.

First, it is assumed that the vertical web reinforcement has not yielded.

Hence:

$$\varepsilon_v = \frac{\varepsilon_r(-\sigma_d + \sigma_v) + \varepsilon_d(-\sigma_v + \sigma_r)}{(-\sigma_d + \sigma_r) - \rho_v E_s (\varepsilon_d - \varepsilon_r)}$$

$$\varepsilon_v = \frac{0.085008(9.968 - 5.163) - 0.000293(5.163 + 2.066)}{(9.968 + 2.066) - 0.0028 \times 200000(-0.000293 - 0.085008)}$$

$$\varepsilon_v = 0.006795$$

ε_v is greater than the lesser value of $(0.8\varepsilon_{yv}; 0.0025)$, therefore ε_v is recalculated as:

$$\varepsilon_v = \varepsilon_r + \left(\frac{-\sigma_v + \sigma_r + \rho_v f_v}{-\sigma_d + \sigma_r} \right) (\varepsilon_d - \varepsilon_r)$$

$$\varepsilon_v = 0.085008 + \left(\frac{5.163 + 2.066 + 0.0028 \times 488}{9.968 + 2.066} \right) (-0.000293 - 0.085008)$$

$$\varepsilon_v = 0.024081$$

$$\varepsilon_h = \varepsilon_d + \varepsilon_r - \varepsilon_v$$

$$\varepsilon_h = -0.000293 + 0.085008 - 0.024081$$

$$\varepsilon_h = 0.060634$$

6. Calculate f_v and f_h using equations (5.10) and (5.11), respectively.

$$f_v = \min(0.8f_{yv}; 500\text{MPa})$$

$$f_v = 488 \text{ MPa}$$

$$f_h = \min(0.8f_{yh}; 500\text{MPa})$$

$$f_h = 488 \text{ MPa}$$

7. Check the equilibrium conditions using equation (5.18).

$$\sigma_v + \sigma_h = \sigma_d + \sigma_r + \rho_v f_v + \rho_h f_h$$

$$-5.163 + 0 = -9.968 + 2.066 + 0.0028 \times 488 + 0.0028 \times 488$$

$$-5.163 = -5.169$$

The total value of left hand side is sufficiently close to the total value of right hand side, thus the equilibrium conditions are satisfied and the assumed value of ε_r is correct. Therefore, subsequent iteration is not needed. If the equilibriums conditions are not satisfied, then go back to Step 3 and assume another value of ε_r (use 0.000001 increment).

8. Calculate θ using equation (5.15).

$$\theta = \sin^{-1} \left(\sqrt{\frac{-\sigma_v + \sigma_r + \rho_v f_v}{-\sigma_d + \sigma_r}} \right)$$

$$\theta = \sin^{-1} \left(\sqrt{\frac{5.163 + 2.066 + 0.0028 \times 488}{9.968 + 2.066}} \right)$$

$$\theta = 57.687^\circ$$

9. Calculate D_u using equation (5.19).

$$b_f = 500 \text{ mm} < 0.5 H_w + t_w = 1100 \text{ mm (OK)}$$

$$A_{sb} = \rho_b \times b_f \times t_f$$

$$A_{sb} = 0.0388 \times 500 \times 120$$

$$A_{sb} = 2328 \text{ mm}^2$$

$$d_{be} = \sqrt{\frac{A_{sb}}{0.25\pi}}$$

$$d_{be} = \sqrt{\frac{2328}{0.25\pi}}$$

$$d_{be} = 54.44 \text{ mm}$$

$$D_u = 1.64b_{ef}d_{be}\sqrt[3]{f'_c}$$

$$D_u = 1.64 \times (500 - 54.44) \times 54.44 \times \sqrt[3]{103.3}$$

$$D_u = 186.65 \text{ kN}$$

10. Calculate V_n using equation (5.20).

$$V_n = (\sigma_r - \sigma_d) \sin \theta \cos \theta t_w d_w + 1.64b_{ef}d_{be}\sqrt[3]{f'_c}$$

$$V_n = \frac{[(2.066 + 9.968) \sin 57.687^\circ \cos 57.687^\circ \times 100 \times 880]}{1000} + 186.65$$

$$V_n = 478.42 + 186.65$$

$$V_n = 665.07 \text{ kN}$$

$$\text{Thus, } V_{exp}/V_n = 595.76/665.07 = 0.90$$

5.3 Simplified Truss Model for Predicting RC Wall Shear Strength

As one of the objectives of this study is to develop an analytical model for predicting RC wall shear strengths that can be used directly by engineers, in this section, the truss model that has been developed based on softened truss model (Hsu 1988) is simplified further to eliminate the need of numerical iterations to obtain the value of ϵ_r (principal strain in r-axis).

5.3.1 Equilibrium Conditions

In this simplified model, the equilibrium conditions that have been derived previously and are expressed by equations (5.2), (5.4), and (5.6) are also applicable here. Unlike the complete truss model that has been developed previously in section 5.2, the simplified truss model does not consider compatibility conditions. This is done in order to simplify the

method so that there is no need to do iterations to find the numerical solutions. This is important since one of the objectives of this study is to develop an accurate analytical method that can be used directly by engineers to predict the shear strength of RC walls. In addition, similar to the complete truss model, in this simplified model, the softening behavior of cracked concrete is also taken into account when determining the capacity of diagonal concrete struts in the wall panel using formulation as proposed by Hsu and his colleagues (Belarbi and Hsu 1994; Belarbi and Hsu 1995; Pang and Hsu 1995; Zhang and Hsu 1998).

5.3.2 Conditions at Ultimate Load Stage

The overall shear strength of RC wall according to this simplified model is governed by either web reinforcement yielding or diagonal concrete strut crushing. The procedure to calculate the shear strength is described as follows. Imposing equilibriums in vertical and horizontal axes of the wall, equations (5.2) and (5.4) can be combined to become equation (5.21). By putting the stress in the diagonal concrete strut (σ_d) to the left hand side and the rest to the right hand side, equation (5.21) can be rearranged as equation (5.22).

$$\sigma_v + \sigma_h = \sigma_d + \sigma_r + \rho_v f_v + \rho_h f_h \quad (5.21)$$

$$-\sigma_d = -\sigma_v - \sigma_h + \sigma_r + \rho_v f_v + \rho_h f_h \quad (5.22)$$

At ultimate load stage, either diagonal concrete strut or web reinforcement will reach their capacities. Hence, a verification needs to be done to see which failure mode governs the overall shear strength of RC wall. Firstly, known terms such as σ_v , σ_h , and σ_r can be replaced by their respective values. σ_v is the applied normal stress in vertical axis caused by external axial force (P/A_g) and the value is positive for tension and negative for compression. The value of σ_h is mostly zero in typical RC wall. σ_r which is the principal stress of concrete in r-axis can be replaced by average residual tensile stress in diagonal concrete strut as

expressed by equation (5.12b) since in the ultimate load stage, the concrete has already cracked. This tensile stress is to take into account the stiffening of steel bars by concrete in tension.

5.3.3 Determination of Failure Modes

After replacing some terms with their respective values, the number of unknown variables in equation (5.22) is reduced to three, i.e. σ_d , f_v , and f_h . Then, verification is done by replacing σ_d term with the compression capacity of cracked diagonal concrete strut ($-\zeta f'_c$) and f_v and f_h terms with the smaller value between 80% yield strengths of the web reinforcements ($0.8f_{yv}$ and $0.8f_{yh}$) and 500 MPa in order to see which failure mode governs the overall shear strength of the RC wall. It has been found in this experimental study that not all (100%) web reinforcements yielded during testing. Thus, it is assumed that the maximum stresses in web reinforcement are about 80% of their yield strengths. Moreover, it is also assumed that the maximum stresses are limited to 500 MPa for typical shear reinforcement. If the left hand side of equation (5.22) is larger than the right hand side, it means both web reinforcements reach their maximum strengths and the value of σ_d will be determined by the total value of the right hand side terms which is less than the compression capacity of cracked diagonal concrete strut ($-\zeta f'_c$). On the other hand, if the left hand side of equation (5.22) is less than the right hand side, it means diagonal concrete strut crushes. In this case, the following assumption is made in order to calculate the stresses in the web reinforcements. If the height to length ratio of RC wall (H_w/L_w) is less than 1.0, it is assumed that only the vertical web reinforcement reaches its maximum strength, whereas if H_w/L_w is equal to or more than 1.0, it is assumed that only the horizontal web reinforcement reaches its maximum strength. This assumption is supported by data obtained from past experiment on RC walls (Barda et al. 1977) and this experimental study. Thus, by this assumption, the remaining stress in web reinforcement (either f_v or f_h) can be calculated.

5.3.4 Softening Factor of Concrete Struts

Similar to the complete truss model, in this simplified model, the softening factor of concrete (ζ) is taken as defined by Hsu and his colleagues (Belarbi and Hsu 1994; Belarbi and Hsu 1995; Pang and Hsu 1995; Zhang and Hsu 1998) which is shown in equation (5.14). However, in this simplified model, since compatibility conditions are not considered, thus the value of ε_r is approximated as 0.005. This value falls within the typical range of ε_r for RC membrane element subjected to shear (Vecchio and Collins 1986). Therefore, by putting the assumed value of ε_r to equation (5.14), the softening factor of concrete (ζ) for the simplified model can be expressed as equation (5.23).

$$\zeta = \left(\frac{5.8}{\sqrt{f'_c}} \leq 0.9 \right) \left(\frac{1}{\sqrt{3}} \right) \quad (5.23)$$

where:

ζ = softening factor of concrete

f'_c = concrete cylinder compressive strength (in MPa)

5.3.5 Determination of Angle of Strut Inclination and Inclusion of Dowel Action

After all terms in equation (5.22) are determined, angle of diagonal concrete strut with respect to horizontal axis (θ) can be calculated by equation (5.15). Similar to the complete truss model, in this simplified model, dowel action from the reinforced boundary elements is included to the nominal shear strength of RC wall. The inclusion of dowel action is in agreement with experimental findings by Corley et al. (1981) and is also confirmed by the author's own experimental results. The boundary elements with concentrated reinforcements at both ends of walls can increase the shear strengths of RC walls. The dowel force (D_u) can be

calculated using equation (5.19) and the nominal shear strength of RC wall can be calculated using equation (5.20).

5.3.6 Summary

Overall, step by step of the procedure for the simplified truss model is summarized as follows:

- 1) Calculate σ_r and ζ using equations (5.12b) and (5.23), respectively.
- 2) Check whether both web reinforcements reach their maximum strengths or diagonal concrete strut crushes using equation (5.22).
 - a. If both web reinforcements reach their maximum strengths, then calculate the value of σ_d which should be less than the compression capacity of cracked diagonal concrete strut ($-\zeta f'_c$).
 - b. If diagonal concrete strut crushes, then calculate the stresses in web reinforcements. For RC wall with H_w/L_w less than 1.0, assume $f_v = \text{minimum } (0.8f_{yv}; 500 \text{ MPa})$ and calculate f_h . Otherwise, assume $f_h = \text{minimum } (0.8f_{yh}; 500 \text{ MPa})$ and calculate f_v .
- 3) Calculate θ using equation (5.15).
- 4) Calculate D_u using equation (5.19).
- 5) Calculate V_n using equation (5.20).

5.3.7 The Calculation of Shear Strength of RC Wall using the Simplified Truss Model

In this section, an example of RC wall shear strength calculation using the simplified truss model is given. As an example, a specimen from the ones tested in this study is used, i.e. specimen J5. The procedure is given as follows:

Specimen J5 data:

Concrete cylinder compressive strength, $f'_c = 103.3 \text{ MPa}$

Wall gross cross section area, $A_g = 196000 \text{ mm}^2$

Axial load applied at wall, $P = 1012 \text{ kN}$ (compression)

Wall height, $H_w = 2000$ mm

Wall length, $L_w = 1000$ mm

Thickness of wall web, $t_w = 100$ mm

Width of boundary element, $b_f = 500$ mm

Thickness of boundary element, $t_f = 120$ mm

Ratio of vertical reinforcement in boundary element, $\rho_b = 0.0388$

Yield strength of vertical reinforcement in boundary element, $f_{yb} = 630$ MPa

Ratio of vertical web reinforcement, $\rho_v = 0.0028$

Yield strength of vertical web reinforcement, $f_{yv} = 610$ MPa

Ratio of horizontal web reinforcement, $\rho_h = 0.0028$

Yield strength of horizontal web reinforcement, $f_{yh} = 610$ MPa

Experimental wall shear strength, $V_{exp} = 595.76$ kN

Calculation of nominal shear strength (V_n) according to the simplified truss model:

1. Calculate σ_r and ζ using equations (5.12b) and (5.23), respectively.

$$\sigma_r = 0.02f'_c$$

$$\sigma_r = 0.02 \times 103.3$$

$$\sigma_r = 2.07 \text{ MPa}$$

$$\zeta = \left(\frac{5.8}{\sqrt{f'_c}} \leq 0.9 \right) \left(\frac{1}{\sqrt{3}} \right)$$

$$\zeta = \left(\frac{5.8}{\sqrt{103.3}} \leq 0.9 \right) \left(\frac{1}{\sqrt{3}} \right)$$

$$\zeta = 0.33$$

2. Check whether both web reinforcements reach their maximum strengths or diagonal concrete strut crushes using equation (5.22).

$$-\sigma_d \quad -\sigma_v - \sigma_h + \sigma_r + \rho_v f_v + \rho_h f_h$$

$$-(\zeta f'_c) \quad -\left(-\frac{P}{A_g} \right) - 0 + \sigma_r + \rho_v 0.8 f_{yv} + \rho_h 0.8 f_{yh}$$

$$(0.33 \times 103.3) \quad (5.16) - 0 + 2.07 + 0.0028 \times 488 + 0.0028 \times 488$$

$$34.09 > 9.96 \rightarrow \text{both web reinforcements reach maximum strengths}$$

$$\sigma_d = -9.96 \text{ MPa}$$

3. Calculate θ using equation (5.15).

$$\theta = \sin^{-1} \left(\sqrt{\frac{-\sigma_v + \sigma_r + \rho_v f_v}{-\sigma_d + \sigma_r}} \right)$$

$$\theta = \sin^{-1} \left(\sqrt{\frac{5.16 + 2.07 + 0.0028 \times 488}{9.96 + 2.07}} \right)$$

$$\theta = 57.71^\circ$$

4. Calculate D_u using equation (5.19).

$$b_f = 500 \text{ mm} < 0.5 H_w + t_w = 1100 \text{ mm (OK)}$$

$$A_{sb} = \rho_b \times b_f \times t_f$$

$$A_{sb} = 0.0388 \times 500 \times 120$$

$$A_{sb} = 2328 \text{ mm}^2$$

$$d_{be} = \sqrt{\frac{A_{sb}}{0.25\pi}}$$

$$d_{be} = \sqrt{\frac{2328}{0.25\pi}}$$

$$d_{be} = 54.44 \text{ mm}$$

$$D_u = 1.64 b_{ef} d_{be} \sqrt[3]{f'_c}$$

$$D_u = 1.64 \times (500 - 54.44) \times 54.44 \times \sqrt[3]{103.3}$$

$$D_u = 186.65 \text{ kN}$$

5. Calculate V_n using equation (5.20).

$$V_n = (\sigma_r - \sigma_d) \sin \theta \cos \theta t_w d_w + 1.64 b_{ef} d_{be} \sqrt[3]{f'_c}$$

$$V_n = \frac{[(2.07 + 9.96) \sin 57.71^\circ \cos 57.71^\circ \times 100 \times 880]}{1000} + 186.65$$

$$V_n = 478.07 + 186.65$$

$$V_n = 664.72 \text{ kN}$$

$$\text{Thus, } V_{\text{exp}}/V_n = 595.76/664.72 = 0.90$$

5.4 Strut and Tie Model for Predicting RC Wall Shear Strength

In addition to the proposed truss model, another method for predicting RC wall shear strength has been developed in this study based on the strut and tie concept. The proposed strut and tie model is described below.

5.4.1 Equilibrium Conditions

The external and internal forces equilibrium of the model is shown in Figure 5.7 and described in equations (5.24) to (5.26). Initially, a typical RC wall with external axial load (P) and lateral load (V) as shown in Figure 5.7 has reaction forces at the bottom of the wall in terms of horizontal reaction force which is equal to V , vertical reaction force and bending moment which can be represented by a combination of compression force (C) and tension force (T). To simplify the load transfer mechanism, the external axial load (P) is combined together with tension force (T) to become a resultant force (R) at point A. This resultant force (R) and external lateral load (V) are equilibrated by diagonal compression force (D) which together (D , V , P , and T) form the strut and tie model. At point B, this diagonal compression force (D) is equilibrated by the vertical compression force (C) and the horizontal reaction force which is equal to V . Shear failure of the RC wall according to this model occurs when the diagonal compression strut crushes. Thus, nominal RC wall shear strength (V_n) is taken as the shear force corresponds to the capacity of the diagonal compression strut (D_n).

Equilibrium of forces in the RC wall:

$$P + T = R \quad (5.24)$$

$$R = C = D \sin \theta \quad (5.25)$$

$$V = D \cos \theta \quad (5.26)$$

where:

P = applied axial force at wall

T = tension force in the tension tie

- R = resultant force of external axial force and tension force in tension tie
- C = compression force in the compression zone at the bottom of wall
- D = compression force in the diagonal concrete strut
- V = applied shear force at the top of wall
- θ = angle of inclination of diagonal concrete strut with respect to horizontal axis

Figure 5.7 Equilibrium of forces in the RC wall.

5.4.2 Determination of Depth of Compression Zone at the Bottom of Wall

The capacity of the diagonal compression strut (D_n) depends mainly on effective strut strength and strut area. The strut area (A_{str}) is defined as strut width multiplied by strut depth. Strut width is taken as thickness of wall web (t_w) while strut depth (a_s) is taken as the perpendicular projection of the depth of compression zone at the bottom of wall (c) to the strut axis as shown in Figure 5.7.

Initially, the author calculated the depth of compression zone (c) based on flexural theory for member subjected to axial load and bending moment, assuming linear strain distribution along wall cross section as Hwang et al. (2001) did on their softened strut and tie model. However, this assumption led to inaccurate predictions of experimental wall shear strengths since the assumption may no longer be valid for RC low-rise walls (which are governed by shear behaviour rather than flexure behaviour) in which plane sections do not remain plane. Therefore, in this study, a formula to determine the value of c was derived with the aid of nonlinear finite element analysis ATENA (Cervenka 2012).

Firstly, several factors affecting the depth of compression zone were identified. According to flexural theory, these factors are concrete compressive strength (f'_c), amount of flexural reinforcement in boundary element or edge column (A_{sb}), and axial load applied (P). From flexural theory for RC members subjected to axial load and bending moment, the value of c decreases if f'_c increases. On the other hand, increment of A_{sb} or P will cause the value of c to increase. Furthermore, the author added wall height to length ratio (H_w/L_w) as additional factor that could affect the value of c because in similar cases, i.e. deep beams, it could be shown that the value of c increased when beam shear span ratio decreased (Park and Paulay 1975).

Secondly, after identifying factors affecting the value of c and their qualitative relationships, the next step is to define the quantitative relationships between these factors and the value of c . The main purpose is to express the value of c as a function of f'_c , A_{sb} , P , and H_w/L_w . For this

purpose, nonlinear finite element analyses were done using ATENA software (Cervenka 2012) to estimate constant multipliers for each factor. Before performing a parametric study to find the quantitative relationships between c and its affecting factors such as f'_c , A_{sb} , P , and H_w/L_w , some verifications of the RC wall finite element model with experimental results were done to ensure the accuracy of material models used. For this purpose, specimen J7 of this experimental study which represents RC wall with H_w/L_w of 2.0 and specimen S-7 as tested by Gupta and Rangan (1998) which represents RC wall with H_w/L_w of 1.0 were used. Comparisons of envelope curve of the specimens obtained from the respective experiments and finite element analysis (FEA) are presented in Figures 5.8 and 5.9. From the figures, it can be seen that the finite element model used can predict accurately the behaviour of RC walls with various height to length ratios, including predicting the maximum lateral force that can be sustained by the RC walls.

Figure 5.8 Comparison of envelope curve of specimen J7 obtained from experiment and finite element analysis (FEA).

Figure 5.9 Comparison of envelope curve of specimen S-7 (Gupta and Rangan 1998) obtained from experiment and finite element analysis (FEA).

For the parametric study to find the quantitative relationships between c and its affecting factors, a typical wall specimen similar to the ones tested in this experimental study was modelled in ATENA software (Cervenka 2012). The influencing factors mentioned above were varied in each analysis case and the value of c at the ultimate load of each case was measured. For concrete compressive strength, two values were used, i.e. $f'_c = 50$ MPa and $f'_c = 100$ MPa. For amount of flexural reinforcement in boundary elements, two values were used, i.e. $A_{sb} = 1200 \text{ mm}^2$ and $A_{sb} = 2400 \text{ mm}^2$. For axial force applied at wall, three values of axial load ratio were used, i.e. 0.00, 0.10, and 0.20. For height to length ratio of wall, three values were used, i.e. 0.40, 1.00, and 2.00. Lastly, the author also tried to vary the boundary element width (b_f), i.e.

120 mm, 250 mm, and 500 mm in order to see variation of c with respect to b_f values.

Therefore, there were a total of 108 cases and values of c obtained from each analysis case were plotted against varying factors to determine the quantitative relationships. Sample pictures of the analysis results from ATENA at peak lateral force are given in Figures 5.10 to 5.12 while the values of c plotted against varying factors are presented in Figures 5.13 to 5.15. The factors of f'_c and P are combined into one to become P/f'_c since this factor is more commonly used as one parameter.

Figure 5.10 State of normal stress in vertical axis (σ_{yy}) at peak lateral load for specimen with $H_w/L_w = 0.4$, axial load ratio of 0.00, $b_f = 120$ mm, $A_{sb} = 2400 \text{ mm}^2$, and $f'_c = 100 \text{ MPa}$.

Figure 5.11 State of normal stress in vertical axis (σ_{yy}) at peak lateral load for specimen with $H_w/L_w = 1.0$, axial load ratio of 0.10, $b_f = 250$ mm, $A_{sb} = 1200 \text{ mm}^2$, and $f'_c = 50 \text{ MPa}$.

Figure 5.12 State of normal stress in vertical axis (σ_{yy}) at peak lateral load for specimen with $H_w/L_w = 2.0$, axial load ratio of 0.20, $b_f = 500$ mm, $A_{sb} = 2400 \text{ mm}^2$, and $f'_c = 100 \text{ MPa}$.

Figure 5.13 Values of c/L_w obtained from FEA plotted against $P/[f'_c A_w]$.

Notes: *W# ALR0.00 f'c50 pb8%. * can be R, B, and F which represent wall with $b_f = 120$ mm, 250 mm, and 500 mm, respectively. # can be L, S, and T which represent wall with H_w/L_w of 0.40, 1.00, and 2.00, respectively. ALR refers to corresponding axial load ratio, while f'_c and ρ_b refer to concrete compressive strength (MPa) and ratio of flexural reinforcement in boundary element, respectively.

Figure 5.14a Values of c/L_w obtained from FEA plotted against $2A_{sb}/A_w$.

Notes: *W# ALR0.00 f'_c 50 ρ_b 8%. * can be R, B, and F which represent wall with $b_f = 120$ mm, 250 mm, and 500 mm, respectively. # can be L, S, and T which represent wall with H_w/L_w of 0.40, 1.00, and 2.00, respectively. ALR refers to corresponding axial load ratio, while f'_c and ρ_b refer to concrete compressive strength (MPa) and ratio of flexural reinforcement in boundary element, respectively.

Figure 5.14b Values of c/L_w obtained from FEA plotted against $2A_{sb}/A_w$.

Notes: *W# ALR0.00 f'_c 50 ρ_b 8%. * can be R, B, and F which represent wall with $b_f = 120$ mm, 250 mm, and 500 mm, respectively. # can be L, S, and T which represent wall with H_w/L_w of 0.40, 1.00, and 2.00, respectively. ALR refers to corresponding axial load ratio, while f'_c and ρ_b refer to concrete compressive strength (MPa) and ratio of flexural reinforcement in boundary element, respectively.

Figure 5.14c Values of c/L_w obtained from FEA plotted against $2A_{sb}/A_w$.

Notes: *W# ALR0.00 f'_c 50 ρ_b 8%. * can be R, B, and F which represent wall with $b_f = 120$ mm, 250 mm, and 500 mm, respectively. # can be L, S, and T which represent wall with H_w/L_w of 0.40, 1.00, and 2.00, respectively. ALR refers to corresponding axial load ratio, while f'_c and ρ_b refer to concrete compressive strength (MPa) and ratio of flexural reinforcement in boundary element, respectively.

Figure 5.15 Values of c/L_w obtained from FEA plotted against H_w/L_w .

Notes: *W# ALR0.00 f'_c 50 ρ_b 8%. * can be R, B, and F which represent wall with $b_f = 120$ mm, 250 mm, and 500 mm, respectively. # can be L, S, and T which represent wall with H_w/L_w of 0.40, 1.00, and 2.00, respectively. ALR refers to corresponding axial load ratio, while f'_c and ρ_b refer to concrete compressive strength (MPa) and ratio of flexural reinforcement in boundary element, respectively.

From Figures 5.13 and 5.14, in general, it can be seen that value of c increases linearly with increment of P/f_c and A_{sb} . On the other hand, value of c decreases exponentially with increment of H_w/L_w (see Figure 5.15). These results are in agreement with qualitative relationships mentioned before. Hence, the value of c can be expressed as in equation (5.27). In this equation, there are four constants that have to be determined. Constants c_2 , c_3 , and c_4 can be obtained from Figures 5.13, 5.14, and 5.15, respectively by plotting regression lines of each data series. From the equations of the regression lines, the constant multipliers are averaged from a total of 108 values. The average regression lines are presented in Figure 5.16. From the figure, the values of c_2 , c_3 , and c_4 are determined as 0.50, 6.00, and -0.40, respectively.

Then, the value of c_1 is determined by trial and error approach to get the most appropriate value. The value of c_1 that results on values of c that best fit to the values of c from nonlinear finite element analysis is 0.35.

Furthermore, from the nonlinear finite element analysis, the author observed that the value of c does not need to be taken greater than effective depth of wall (d_w) that is defined as the distance of center to center of wall boundary elements or $0.8L_w$ in case of wall without boundary elements.

Finally, the depth of compression zone at the bottom of wall (c) can be calculated using equation (5.27) with constants c_{1-4} as defined above. In addition, a simple verification is also done to check the accuracy of the values of c calculated using equation (5.27) with the values of c obtained from FEA. The comparison of 108 data shows that in average, the normalized values of c obtained from FEA over values of c calculated using equation (5.27) is 1.07 with minimum and maximum values of 0.93 and 1.23, respectively. Moreover, the standard deviation and coefficient of variation values are 0.07 and 0.07, respectively. Therefore, it can be concluded that the predictions of c values using equation (5.27) are quite close to the values of c obtained from FEA.

$$c = L_w \left(c_1 + c_2 \frac{P}{f'_c A_w} + c_3 \frac{A_{sb}}{A_w} \right) \left(\frac{H_w}{L_w} \right)^{c_4} \leq d_w \quad (5.27)$$

where:

c = depth of compression zone at the bottom of wall (mm)

c_{1-4} = constants to be determined

P = applied axial force at wall (N)

f'_c = concrete cylinder compressive strength (MPa)

A_w = area of wall web (mm²)

A_{sb} = area of flexural reinforcement in boundary element (mm²)

H_w = wall height (mm)

L_w = wall length (mm)

d_w = effective depth of wall (mm)

Figure 5.16 The average regression lines and their equations.

5.4.3 Capacity of Diagonal Compression Strut

The capacity of the diagonal compression strut (D_n) depends mainly on effective strut strength ($\zeta f'_c$) and strut area (A_{str}) as presented in equation (5.28). The explanation to obtain A_{str} has been given in previous

section. In this model, effective strut strength is taken as recommended by Eurocode 2 (2004). The recommendation takes into account a reduction factor of concrete strut strength because of tensile stresses on concrete which cause cracks in the strut. Furthermore, in this model, confinement effect to concrete strut strength by means of appropriate amount of transverse reinforcement is also taken into account as recommended by FIP Commission 3 (1999). The implication of these factors into the effective strut strength in this model is described in equations (5.29) to (5.31).

The definition of transverse reinforcement here refers to the reinforcement that is perpendicular to the strut axis and provides confinement effect to the concrete strut. Hence, it is necessary to represent vertical and horizontal web reinforcement of the RC wall to be transverse reinforcement of the concrete strut as defined by FIP Commission 3 (1999). In this model, the term ρf_y is represented as equation (5.32). Furthermore, in this model, the value of θ is limited to $31^\circ \leq \theta \leq 59^\circ$. Lastly, the nominal shear strength of RC wall (V_n) according to this model can be calculated by equation (5.34).

$$D_n = \zeta f'_c A_{str} \quad (5.28)$$

$$\zeta = 0.6 \left(1 - \frac{f'_c}{250} \right) \times 0.80 (1 + 1.6 \alpha_w \omega_w) \leq 0.85 \quad (5.29)$$

$$\alpha_w = 1.6 \frac{s}{t_w} \leq 0.4 \quad (5.30)$$

$$\omega_w = 4 \frac{\rho f_y}{f'_c} \quad (5.31)$$

$$\rho f_y = \rho_v f_{yv} \cos \theta + \rho_h f_{yh} \sin \theta \quad (5.32)$$

$$\theta = \tan^{-1} \left(\frac{H'}{L_w - r - 0.5c} \right) \quad (5.33)$$

$$V_n = D_n \cos \theta \quad (5.34)$$

where:

D_n = nominal strength of diagonal concrete strut (N)

- ζ = softening factor of concrete
- f'_c = concrete cylinder compressive strength (MPa)
- A_{str} = area of diagonal concrete strut (mm^2)
- α_w = coefficient taking account of confinement effect of web reinforcement to concrete strut strength, related to spacing of web reinforcement
- ω_w = coefficient taking account of confinement effect of web reinforcement to concrete strut strength, related to ratio of web reinforcement
- s = spacing of reinforcement (mm)
- t_w = thickness of wall web (mm)
- ρ = reinforcement ratio
- f_y = yield strength of reinforcement (MPa)
- ρ_v = vertical web reinforcement ratio
- f_{yv} = yield strength of vertical web reinforcement (MPa)
- ρ_h = horizontal web reinforcement ratio
- f_{yh} = yield strength of horizontal web reinforcement (MPa)
- θ = angle of inclination of diagonal concrete strut with respect to horizontal axis
- H' = distance measured from point of application of external shear force to wall base (mm)
- L_w = wall length (mm)
- r = distance measured from point of application of resultant force to nearest wall edge (mm)
- c = depth of compression zone at the bottom of wall (mm)
- V_n = nominal shear strength of RC wall (N)

5.4.4 Summary

Overall, step by step of the procedure is summarized as follows:

- 1) Calculate c using equation (5.27) and the corresponding A_{str} .
- 2) Calculate ζ using equation (5.29).
- 3) Calculate D_n using equation (5.28).

4) Calculate V_n using equation (5.34).

5.4.5 The Calculation of Shear Strength of RC Wall using the Proposed Strut and Tie Model

In this section, an example of shear strength calculation of a RC wall using the proposed strut and tie model is given. As an example, a specimen from the ones tested in this study is used, i.e. specimen J5. The procedure is given as follows:

Specimen J5 data:

Concrete cylinder compressive strength, $f'_c = 103.3$ MPa

Wall gross cross section area, $A_g = 196000$ mm²

Axial load applied at wall, $P = 1012$ kN (compression)

Wall height, $H_w = 2000$ mm

Wall length, $L_w = 1000$ mm

Distance from point of application of external shear force to wall base, $H' = 2200$ mm

Thickness of wall web, $t_w = 100$ mm

Width of boundary element, $b_f = 500$ mm

Thickness of boundary element, $t_f = 120$ mm

Ratio of vertical reinforcement in boundary element, $\rho_b = 0.0388$

Yield strength of vertical reinforcement in boundary element, $f_{yb} = 630$ MPa

Ratio of vertical web reinforcement, $\rho_v = 0.0028$

Yield strength of vertical web reinforcement, $f_{yv} = 610$ MPa

Ratio of horizontal web reinforcement, $\rho_h = 0.0028$

Yield strength of horizontal web reinforcement, $f_{yh} = 610$ MPa

Spacing of web reinforcement, $s = 200$ mm

Experimental wall shear strength, $V_{exp} = 595.76$ kN

Calculation of nominal shear strength (V_n) according to the proposed strut and tie model:

1) Calculate c using equation (5.27) and the corresponding A_{str} .

$$c = L_w \left(0.35 + 0.5 \frac{P}{f'_c A_w} + 6 \frac{A_{sb}}{A_w} \right) \left(\frac{H_w}{L_w} \right)^{-0.4} \leq d_w$$

$$c = 1000 \left(0.35 + 0.5 \frac{1012000}{103.3 \times 100000} + 6 \frac{2328}{100000} \right) \left(\frac{2000}{1000} \right)^{-0.4}$$

$$c = 408.23 \text{ mm} \leq 880 \text{ mm (OK)}$$

Calculating T assuming yielding of reinforcement:

T_1 from vertical reinforcement in boundary element:

$$T_1 = \rho_b \times b_f \times t_f \times f_{yb}$$

$$T_1 = 0.0388 \times 500 \times 120 \times 630$$

$$T_1 = 1466.64 \text{ kN}$$

T_2 from vertical web reinforcement that is in tension:

$$T_2 = \rho_v \times (L_w - c - t_f) \times t_w \times f_{yv}$$

$$T_2 = 0.0028 \times (1000 - 408.23 - 120) \times 100 \times 610$$

$$T_2 = 80.58 \text{ kN}$$

Calculating r by taking wall edge in tension as reference point:

$$r = \frac{T_1 \times \text{arm}_1 + T_2 \times \text{arm}_2 + P \times 0.5L_w}{T_1 + T_2 + P}$$

$$= \frac{1466.64 \times 0.5 \times 120 + 80.58 \times [120 + 0.5 \times (1000 - 408.23 - 120)] + 1012 \times 500}{1466.64 + 80.58 + 1012}$$

$$r = 243.31 \text{ mm}$$

Calculate θ using equation (5.33):

$$\theta = \tan^{-1} \left(\frac{H'}{L_w - r - 0.5c} \right)$$

$$\theta = \tan^{-1} \left(\frac{2200}{1000 - 243.31 - 0.5 \times 408.23} \right)$$

$$\theta = 75.9^\circ$$

Then, take $\theta = 59^\circ$

Calculating A_{str} :

$$A_{str} = a_s \times t_w$$

$$A_{str} = c \times \sin \theta \times t_w$$

$$A_{str} = 408.23 \times \sin 59^\circ \times 100$$

$$A_{str} = 34992.14 \text{ mm}^2$$

2) Calculate ζ using equation (5.29).

Calculating α_w using equation (5.30):

$$\alpha_w = 1.6 \frac{s}{t_w} \leq 0.4$$

$$\alpha_w = 1.6 \frac{200}{100}$$

$$\alpha_w = 3.2$$

Then, take $\alpha_w = 0.4$

Calculating ω_w using equation (5.31):

$$\rho f_y = \rho_v f_{yv} \cos \theta + \rho_h f_{yh} \sin \theta$$

$$\rho f_y = 0.0028 \times 610 \times \cos 59^\circ + 0.0028 \times 610 \times \sin 59^\circ$$

$$\rho f_y = 2.34$$

$$\omega_w = 4 \frac{\rho f_y}{f'_c}$$

$$\omega_w = 4 \frac{2.34}{103.3}$$

$$\omega_w = 0.09$$

$$\zeta = 0.6 \left(1 - \frac{f'_c}{250} \right) \times 0.80 (1 + 1.6 \alpha_w \omega_w) \leq 0.85$$

$$\zeta = 0.6 \left(1 - \frac{103.3}{250} \right) \times 0.80 (1 + 1.6 \times 0.4 \times 0.09)$$

$$\zeta = 0.30$$

3) Calculate D_n using equation (5.28).

$$D_n = \zeta f'_c A_{str}$$

$$D_n = 0.30 \times 103.3 \times 34992.14$$

$$D_n = 1077.18 \text{ kN}$$

4) Calculate V_n using equation (5.34).

$$V_n = D_n \cos \theta$$

$$V_n = 1077.18 \times \cos 59^\circ$$

$$V_n = 554.79 \text{ kN}$$

$$\text{Thus, } V_{exp}/V_n = 595.76/554.79 = 1.07$$

5.5 Evaluation of the Proposed Models

To verify the accuracy of the proposed models, data from past experiments on RC walls failing in shear as well as those obtained from this experimental study as presented in previous chapters are used. Furthermore, shear strength predictions of RC walls using the proposed models are compared with predictions from building codes (Comite Europeen de Normalisation 2004; ACI Committee 318 2014) and other analytical models (Gupta and Rangan 1998; Hwang and Lee 2002). In addition, the normalized experimental wall shear strength by nominal wall shear strength (V_{exp}/V_n) is plotted against height to length ratio, concrete compressive strength, and ratio of vertical reinforcement in boundary element in order to see the variation of predictions as influenced by these parameters. The analysis results are presented in Table 5.1 and Figures 5.17 to 5.19.

From Table 5.1, it can be concluded that overall the proposed models are more accurate than building codes (Comite Europeen de Normalisation 2004; ACI Committee 318 2014) and other analytical models (Gupta and Rangan 1998; Hwang and Lee 2002) in predicting RC wall shear strengths. This is shown by the average predictions V_{exp}/V_n that are closer to 1.00, except for Hwang and Lee (2002), while still being in the conservative side and the coefficient of variation (COV) values that are smaller as compared to other predictions. Moreover, it can be seen that Eurocode 8 (2004) is the most conservative one with average predicted value of V_{exp}/V_n of 2.13 while the analytical model by Gupta and Rangan (1998) has the highest value of COV (0.75). In addition, it is worth noting that the shear strength predictions of the complete truss model (that considers compatibility conditions) and the simplified truss model (that does not consider compatibility conditions) are almost identical. There are only three specimens whose predictions are different. This is because in most cases, it has been found that the web reinforcement reached their maximum strengths first before diagonal concrete strut crushes. Hence, the failure mode governs is web reinforcement reaching their maximum stresses and the equilibrium conditions as presented by equation (5.22) will be limited to the total value of the right hand side equation.

This means that the assumptions taken to simplify the truss model are quite reasonable by keeping the method simple enough (does not need to perform numerical iterations) without losing its accuracy.

Table 5.1 Values of normalized experimental wall shear strength.

No.	Specimen ID	V _{exp} /V _n						
		ACI 318	EC8	Hwang-Lee	Gupta-Rangan	Complete Truss Model	Simplified Truss Model	Proposed Strut-Tie Model
(Barda et al. 1977)								
1	B1-1	1.65	3.94	1.23	0.98	2.11	2.11	1.52
2	B2-1	1.51	3.45	1.72	1.17	1.70	1.70	1.39
3	B3-2	1.48	3.23	1.18	0.91	1.80	1.80	1.29
4	B6-4	1.25	2.72	1.39	1.50	1.92	1.92	1.33
5	B7-5	1.56	4.64	1.09	1.16	2.18	2.18	1.11
6	B8-5	1.24	2.24	1.82	1.66	1.46	1.46	1.57
(Cardenas et al. 1980)								
7	SW-7	1.30	2.06	0.88	1.11	1.54	1.54	1.03
8	SW-8	1.36	2.02	0.97	0.37	0.93	0.93	0.96
(Corley et al. 1981)								
9	B2	0.76	1.31	1.04	5.73	1.15	1.15	1.04
10	B5	0.91	1.56	1.27	6.77	1.41	1.41	1.30
11	B6	1.10	1.96	1.56	2.58	1.21	1.21	1.78
12	B7	1.18	2.05	1.11	2.65	1.17	1.17	1.40
13	B8	0.94	1.38	1.13	2.69	0.92	0.92	1.31
14	B9	1.25	2.17	1.12	2.68	1.23	1.23	1.49
15	B10	0.90	1.56	0.81	1.94	0.90	0.90	1.17
16	F1	0.90	1.45	1.41	6.19	0.99	0.99	1.51
17	F2	1.13	1.96	0.91	2.31	0.79	0.79	1.28
(Maeda 1986)								
18	MAE03	1.46	2.82	1.02	0.81	1.69	1.69	1.09
19	MAE07	1.52	2.38	1.10	0.68	1.40	1.40	1.11
(Okamoto 1990)								
20	W48M6	1.10	1.99	0.88	0.88	1.13	1.13	1.16
21	W48M4	1.12	1.97	0.86	0.90	1.13	1.13	1.14
22	W72M8	1.33	1.89	1.20	0.83	1.35	1.35	1.41
23	W72M6	1.30	1.93	1.17	0.86	1.18	1.18	1.38

Table 5.1 Values of normalized experimental wall shear strength (continued).

No.	Specimen ID	V_{exp}/V_n						
		ACI 318	EC8	Hwang-Lee	Gupta-Rangan	Complete Truss Model	Simplified Truss Model	Proposed Strut-Tie Model
24	W72M8	1.23	1.93	1.14	0.86	1.31	1.31	1.40
25	W96M8	1.44	2.04	1.33	0.81	1.30	1.30	1.49
(Mo and Chan 1996)								
26	HN4-1	0.88	1.58	0.87	0.91	1.35	1.35	0.87
27	HN4-2	1.06	1.90	1.05	1.10	1.63	1.63	1.05
28	HN4-3	0.87	1.56	0.86	0.90	1.33	1.33	0.86
29	HN6-1	0.94	1.70	1.18	0.77	1.30	1.30	1.04
30	HN6-2	0.75	1.36	0.95	0.62	1.04	1.04	0.83
31	HN6-3	0.74	1.31	0.90	0.62	1.04	1.04	0.81
32	HM4-1	0.93	1.69	0.81	1.00	1.41	1.41	0.86
33	HM4-2	0.96	1.75	0.84	1.04	1.46	1.46	0.89
34	HM4-3	1.03	1.88	0.86	1.12	1.55	1.55	0.93
35	LN4-1	0.91	2.00	1.47	1.04	1.57	1.57	1.21
36	LN4-2	1.02	2.25	1.65	1.17	1.76	1.76	1.36
37	LN4-3	0.88	1.59	0.93	1.12	1.47	1.47	0.92
38	LN6-1	0.89	1.58	1.10	0.93	1.35	1.35	1.00
39	LN6-2	0.73	1.30	0.91	0.76	1.10	1.10	0.82
40	LN6-3	0.76	1.37	0.95	0.80	1.16	1.16	0.86
41	LM6-1	0.70	1.28	0.76	0.84	1.14	1.14	0.77
42	LM6-2	0.67	1.21	0.76	0.78	1.08	1.08	0.75
43	LM6-3	0.72	1.24	0.83	0.80	1.12	1.12	0.80
44	LM4-2	0.92	1.78	0.69	1.40	1.37	1.37	0.69
45	LM4-3	0.84	1.62	0.63	1.27	1.24	1.24	0.63
(Gupta and Rangan 1998)								
46	S-1	1.11	1.58	0.99	1.12	1.07	1.07	1.03
47	S-2	1.96	2.24	1.32	1.03	1.46	1.46	1.55
48	S-3	2.28	2.28	1.23	0.88	1.43	1.43	1.53
49	S-4	1.58	2.16	1.43	1.07	1.30	1.30	1.32
50	S-5	2.10	2.43	1.42	0.90	1.41	1.41	1.49
51	S-6	2.59	2.60	1.40	0.94	1.53	1.52	1.62
52	S-7	1.52	2.05	1.41	1.15	1.32	1.32	1.56

Table 5.1 Values of normalized experimental wall shear strength (continued).

No.	Specimen ID	V _{exp} /V _n						
		ACI 318	EC8	Hwang-Lee	Gupta-Rangan	Complete Truss Model	Simplified Truss Model	Proposed Strut-Tie Model
(Kabeyasawa and Hiraishi 1998)								
53	W-08	1.48	1.93	1.35	1.10	1.62	1.62	1.89
54	W-12	1.46	1.95	1.21	0.95	1.40	1.40	1.99
55	No. 1	2.25	2.19	1.11	1.04	1.70	1.70	1.48
56	No. 2	1.90	1.93	1.18	1.06	1.61	1.61	1.55
57	No. 3	1.60	1.84	1.23	1.03	1.51	1.51	1.59
58	No. 4	1.84	1.88	1.22	0.94	1.42	1.42	1.70
59	No. 5	1.41	1.50	1.07	1.31	1.25	1.25	1.55
60	No. 6	1.45	1.86	1.26	1.01	1.40	1.40	1.34
61	No. 7	1.57	2.01	1.34	1.00	1.22	1.22	1.50
62	No. 8	1.66	2.13	1.45	1.01	1.08	1.07	1.45
(Farvashany et al. 2008)								
63	HSCW1	2.20	2.36	1.56	1.34	1.41	1.41	1.62
64	HSCW2	2.60	2.48	1.60	1.18	1.52	1.52	1.78
65	HSCW3	1.96	1.85	1.19	1.07	1.22	1.22	1.38
66	HSCW4	2.68	1.99	1.13	0.84	1.32	1.28	1.56
67	HSCW5	1.93	2.07	1.42	1.12	1.32	1.32	1.66
68	HSCW6	1.77	1.94	1.49	1.35	1.34	1.34	1.63
69	HSCW7	1.85	1.94	1.39	1.37	1.34	1.34	1.67
(Burgueno et al. 2014)								
70	M05C	1.85	2.68	2.46	3.05	1.43	1.43	1.62
71	M05M	2.14	3.23	2.76	3.46	1.55	1.55	1.81
72	M10C	1.56	2.19	2.22	2.73	1.24	1.24	1.39
73	M10M	1.53	2.09	2.43	3.27	1.39	1.39	1.51
74	M15C	1.27	1.77	2.09	2.96	1.21	1.21	1.37
75	M15M	1.38	1.98	2.33	3.39	1.35	1.35	1.54
76	M20C	1.11	1.72	1.92	3.13	1.08	1.08	1.35
77	M20M	1.34	1.95	2.27	3.55	1.26	1.26	1.49
Author								
78	J1	2.85	3.25	1.62	1.93	1.82	1.82	1.65
79	J2	3.05	3.48	1.75	1.52	1.83	1.83	1.71
80	J3	2.09	2.36	1.71	2.21	1.77	1.77	1.87

Table 5.1 Values of normalized experimental wall shear strength (continued).

No.	Specimen ID	V_{exp}/V_n						
		ACI 318	EC8	Hwang-Lee	Gupta-Rangan	Complete Truss Model	Simplified Truss Model	Proposed Strut-Tie Model
81	J4	1.97	2.35	1.44	1.71	2.07	2.07	1.19
82	J5	1.73	4.36	1.07	1.92	0.90	0.90	1.07
83	J6	2.14	5.30	1.33	1.75	1.04	1.04	1.29
84	J7	1.46	2.58	1.23	2.74	1.09	1.09	1.52
Statistical Parameters								
Minimum Value		0.67	1.21	0.63	0.37	0.79	0.79	0.63
Maximum Value		3.05	5.30	2.76	6.77	2.18	2.18	1.99
Average		1.43	2.13	1.29	1.59	1.36	1.36	1.31
Standard Deviation		0.54	0.74	0.43	1.19	0.28	0.28	0.32
Coefficient of Variation		0.38	0.35	0.33	0.75	0.20	0.20	0.24

In Figures 5.17 to 5.19, STM refers to the simplified truss model while PST refers to the proposed strut-tie model. Here, since the complete truss model predictions are almost identical to the simplified truss model predictions, the value of V_{exp}/V_n plotted is presented only for the simplified truss model. In the figures, the trend lines plotted represent the average values of V_{exp}/V_n for certain ranges of parameters. They are plotted in order to see the accuracy of RC wall shear strength predictions of the methods for different range of parameters. From the figures, in general, it seems that the STM predictions are uniformly accurate for various ranges of parameters plotted and they are less scattered as compared to predictions by other methods. The trend lines of the STM predictions do not vary much with variation of the parameters. On the other hand, the trend lines of predictions by other methods seem to vary with the parameters which implies the non-uniform accuracies of their predictions for certain ranges of parameters plotted. Comparing STM and PST predictions, STM is less conservative for RC walls with $H_w/L_w > 2.0$ while PST is less conservative for RC walls with $H_w/L_w < 1.0$. In addition, STM is uniformly conservative for RC walls with all ranges of f'_c and ρ_b while PST is less conservative for RC walls with $f'_c < 50$ MPa and for RC walls with $\rho_b < 0.03$.

Figure 5.17 Normalized experimental wall shear strength plotted against height to length ratio.

Figure 5.18 Normalized experimental wall shear strength plotted against concrete compressive strength.

Figure 5.19 Normalized experimental wall shear strength plotted against ratio of vertical reinforcement in boundary element.

5.5.1 Further Discussions on ACI 318

For ACI 318 code (2014), it is worth noting that the predictions may not be conservative enough for RC walls with concrete compressive strength below 50 MPa whereas they are more conservative for RC walls with concrete compressive strength greater than 50 MPa. For HSC walls tested in this study, V_{exp}/V_n values from ACI 318 code (2014) predictions are 2.85, 3.05, 2.09, 1.97, 1.73, 2.14, and 1.46 for specimen J1, J2, J3, J4, J5, J6, and J7, respectively. Further investigation shows that the nominal wall shear strengths calculated as ACI 318 equation (18.10.4.1) as shown in equation (2.21) herein are too small as compared to experimental wall shear strengths for these specimens (see Table 5.1 and Figure 5.18). This can be due to contribution of vertical web reinforcement to overall shear strength of RC wall is not taken into account in the ACI 318 code (2014). From experiment results in this study, it has been found that vertical web reinforcement also has contribution to the shear strength of RC walls. Thus, the inaccuracies are larger for specimens J2 and J6 (with more vertical web reinforcement) as compared to specimens J1 and J5.

5.5.2 Further Discussions on Eurocode 8

Eurocode 8 (2004) underestimates wall shear strengths by significant margin for walls having low height to length ratio, especially those tested by Barda et al. (1977) that had height to length ratio of 0.25 and 0.50. Barda et al. (1977) concluded that horizontal web reinforcement did not contribute to shear strength for such low rise walls. Instead, they observed that actually vertical web reinforcement was effective as shear reinforcement for those specimens. In Eurocode 8 (2004), the equation (5.38) as shown in equation (2.24) herein for calculating diagonal tension capacity of low rise walls has covered the declining contribution of horizontal web reinforcement to wall shear strength with reduction in height to length ratio. However, it does not include the contribution of vertical web reinforcement to wall shear strength which is actually effective for such low rise walls. Therefore, the code underestimates wall

shear strengths by significant margin. Furthermore, for HSC walls tested in this study, Eurocode 8 (2004) also significantly underestimates the shear strengths with V_{exp}/V_n values of 3.25, 3.48, 2.36, 2.35, 4.36, 5.30, and 2.58 for specimen J1, J2, J3, J4, J5, J6, and J7, respectively. The prediction inaccuracies are larger for walls with higher height to length ratio because according to Eurocode 8 (2004), for walls with moment to shear ratio equal or more than 2.0, it only takes into account contribution of horizontal web reinforcement to overall shear strength of RC wall while the contributions of the concrete and the vertical web reinforcement are neglected.

5.5.3 Further Discussions on Analytical Model by Hwang and Lee (2002) and the Proposed Strut-Tie Model

The analytical model by Hwang and Lee (2002) has the closest to 1.00 of average predictions value (V_{exp}/V_n) among other methods. However, the coefficient of variation value is still larger as compared to the complete and simplified truss models and the proposed strut-tie model (see Table 5.1). Comparing the analytical model by Hwang and Lee (2002) with the proposed strut-tie model, the proposed strut-tie model offers different shear resisting mechanism (utilizing one diagonal strut) which is simpler as compared to the analytical model by Hwang and Lee (2002) that utilizes three struts. Furthermore, the proposed strut-tie model takes into account the effect of axial loading explicitly into the strut and tie forces whereas the analytical model by Hwang and Lee (2002) treats it indirectly as additional vertical web reinforcement. Moreover, the proposed strut-tie model uses equation (5.27) that is developed based on nonlinear finite element analysis in order to determine the depth of compression zone at the bottom of wall whereas the analytical model by Hwang and Lee (2002) calculates it based on flexural theory assuming linear strain distribution along wall cross section. This assumption of linear strain distribution may not be correct since for squat walls whose behavior is dominated by shear, it is known that plane section does not

remain plane. Therefore, it can be concluded that the proposed strut-tie model provides simpler shear resisting mechanism and yet more accurate RC wall shear strength predictions as compared to the analytical model by Hwang and Lee (2002). Nevertheless, the predictions of both models are not conservative enough for some specimens in the database, especially those tested by Mo and Chan (1996) that have low height to length ratio (less than 1.0).

5.5.4 Further Discussions on Analytical Model by Gupta and Rangan (1998) and the Simplified Truss Model

The analytical model by Gupta and Rangan (1998) has the highest value of coefficient of variation as compared to other methods. This is because this model significantly underestimates the shear strength of RC walls with height to length ratio more than 2.0 (see Figure 5.17). This may be due to inaccuracy in determining the strut angle, which in this model, it is only a function of effective depth of wall and wall height. In contrast, for the simplified truss model, the strut angle is calculated using equation (5.15) which takes into account the effect of axial loading, web reinforcement ratios, etc. and thus it results in better predictions of the shear strength of RC walls. Furthermore, as can be seen in Figure 5.19, the simplified truss model that includes dowel action of reinforced boundary elements has uniform accuracy of RC wall shear strength predictions regardless the value of ρ_b whereas the analytical model by Gupta and Rangan (1998) is less accurate as the value of ρ_b increases. This clearly shows that it is important to include dowel action of reinforced boundary element in order to predict more accurately the shear strength of RC walls.

5.6 Conclusions

In conclusions, analytical models based on the truss analogy have been developed to predict the shear strength of RC walls, i.e. the complete and simplified truss models and the proposed strut-tie model. In the complete truss

model, equilibrium conditions, compatibility conditions, and the softening behavior of cracked concrete have been considered while in the simplified truss model, only equilibrium conditions and the softening behavior of cracked concrete are taken into account. This is done so that there is no need to perform numerical iterations to find the solutions and hence the simplified model can be used directly by engineers to predict the shear strength of RC walls.

As learned from this experimental study, in the analytical models, contribution from both vertical and horizontal web reinforcements are included to overall shear strength of RC walls and their effectiveness is determined by wall height to length ratio. In addition, the contribution of dowel action from reinforced boundary element is included as well to the shear strength of RC walls. An alternate method utilizing strut and tie concept is also developed. The proposed strut-tie model offers different shear resisting mechanism which is simpler as compared to the analytical model by Hwang and Lee (2002). Furthermore, the proposed strut-tie model is also able to take into account the effect of axial loading explicitly into the strut and tie forces. In addition, a formula to calculate the depth of compression zone at the bottom of wall has also been developed based on nonlinear finite element analysis. This is done since the depth of compression zone calculated from the flexural theory assuming linear strain distribution along wall cross section does not result in accurate prediction of RC wall shear strengths. In the formula, several key factors that affect the depth of compression zone have been considered, such as axial loading, concrete compressive strength, amount of vertical reinforcement in boundary element, and wall height to length ratio.

The predictions of the proposed models are verified with a total of 84 RC wall specimens failing in shear from available literature as well as from this experimental study. The predictions are also compared with the predictions from building codes (Comite Europeen de Normalisation 2004; ACI Committee 318 2014) and other analytical models (Gupta and Rangan 1998; Hwang and Lee 2002). It is worth noting that shear strength predictions of the complete truss model and the simplified truss model are almost identical. This is due to in most cases, the failure mode that governs is web reinforcement

reaching their maximum stresses rather than diagonal concrete strut crushing. Thus, the assumptions taken in order to simplify the truss model are quite reasonable by keeping the method simple enough (does not need to perform numerical iterations) without losing its accuracy. The analysis results show that the predictions of the proposed models are in average closer to experimental shear strengths of RC walls while still being on the conservative side. Moreover, the overall predictions of the proposed models have lower coefficient of variation compared to those of other prediction methods. In addition, the complete and simplified truss models can predict the shear strength of RC walls with consistent accuracy for wide ranges of wall height to length ratios and concrete compressive strengths. This is not the case for the predictions from building codes (Comite Europeen de Normalisation 2004; ACI Committee 318 2014) and other analytical models (Gupta and Rangan 1998; Hwang and Lee 2002). Their predictions show non-uniform accuracies for certain ranges of wall height to length ratios and concrete compressive strengths. Furthermore, in their formulation for calculating the shear strength of RC walls, the contribution of dowel action from reinforced boundary element is not included and hence their predictions are also less accurate for RC walls with high ratio of vertical reinforcement in boundary element.

CHAPTER 6

CONCLUSIONS AND FUTURE WORKS

6.1 Conclusions

In this chapter, overall conclusions regarding this study are described. While conclusions regarding experimental study and analytical models are already described in previous chapters, in this chapter, several key findings are highlighted.

From the experimental study, seven HSC wall specimens with concrete compressive strength about 100 MPa or higher have been tested under static vertical axial loading and cyclic lateral in-plane loading until failure to investigate their shear behaviour. The main parameters are wall height to length ratio, vertical and horizontal web reinforcement ratios, and presence of boundary elements.

All HSC wall specimens failed in brittle shear mode as expected. The hysteretic behaviour of the specimens is typical behaviour of RC wall failing in shear with severe strength degradation after reaching its ultimate strength. In this regard, it can be concluded that the hysteretic behaviour of HSC walls failing in shear does not differ much with NSC walls failing in shear. Moreover, the wall drift at peak load is mostly below 1.0% for all specimens which is also in typical range for RC wall failing in shear.

The effect of wall height to length ratio is highlighted with respect to wall shear strength. Together with the results from previous experiment by Barda et al. (1977), it can be concluded that every increment of 0.5 in wall height to length ratio (for height to length ratio ranging from 0.5 to 2.0), the overall wall shear strength will be reduced by 20%. Further investigation shows that this reduction of wall shear strength is mainly because the concrete contribution to wall shear strength is greatly reduced.

The effect of vertical and horizontal web reinforcement is also highlighted with respect to wall shear strength. From this experimental study, it can be concluded that horizontal web reinforcement is more effective than vertical web reinforcement in contributing to wall shear strength for walls with height

to length ratio of 1.0 and 2.0. In addition, previous experiment by Barda et al. (1977) concluded that the vertical web reinforcement is more effective for wall with height to length ratio less than 1.0. Therefore, it can be concluded that for wall with height to length ratio less than 1.0, vertical web reinforcement is more effective than horizontal web reinforcement in contributing to wall shear strength while the opposite is true for wall with height to length ratio equal to or greater than 1.0. Another key finding from this experimental study is that contribution of web reinforcement (especially horizontal one) to overall wall shear strength increases with increment of wall height to length ratio while concrete contribution decreases.

The presence of boundary elements can increase significantly wall shear strength. From this experimental study, it has been found that for walls with boundary elements, the normalized shear strength is about 40% greater than that of wall without boundary elements. This finding is in agreement with experiment conducted by Corley et al. (1981).

Building code provisions (Comite Europeen de Normalisation 2004; ACI Committee 318 2014) significantly underestimate the shear strength of HSC walls tested in this study. This is because the codes neglect the contributions of vertical web reinforcement and dowel action of reinforced boundary element to the shear strength of RC walls. In this experimental study, it is found that these two factors indeed have considerable contributions to the shear strength of RC walls.

In conclusions, the results from this experimental study provide useful data which combined with the results from other experiments, they can enhance current understanding about overall shear behaviour of RC walls, especially HSC walls and factors affecting it.

For the analytical work, in this study, analytical models based on the truss analogy have been developed to predict the shear strength of RC walls. In the complete truss model, both equilibrium and compatibility conditions are considered whereas in the simplified truss model, only equilibrium equations are taken into account. This is done so that there is no need to perform numerical iterations to find solutions and hence the simplified model can be

used directly by engineers to predict the shear strength of RC walls. As learned from this experimental study, in the analytical models, contribution from both vertical and horizontal web reinforcements are included to overall shear strength of RC walls and their effectiveness is determined by wall height to length ratio. In addition, the contribution of dowel action from reinforced boundary element is included as well to the shear strength of RC walls.

An alternate method utilizing strut and tie concept to predict the shear strength of RC walls is also developed in this study. The proposed strut-tie model offers simpler shear resisting mechanism as compared to the analytical model by Hwang and Lee (2002). Furthermore, the proposed strut-tie model is able to take into account the effect of axial loading explicitly into the strut and tie forces. In addition, a formula to calculate the depth of compression zone at the bottom of wall has also been developed based on nonlinear finite element analysis. This is done since the depth of compression zone calculated from the flexural theory assuming linear strain distribution along wall cross section does not result in accurate prediction of RC wall shear strengths. In the formula, several key factors that affect the depth of compression zone have been considered, such as axial loading, concrete compressive strength, amount of vertical reinforcement in boundary element, and wall height to length ratio.

The proposed model predictions are verified with a total of 84 RC wall specimens failing in shear from available literature as well as from this experimental study. Moreover, the predictions from the proposed models are compared with the predictions from building codes (Comite Europeen de Normalisation 2004; ACI Committee 318 2014) and other analytical models (Gupta and Rangan 1998; Hwang and Lee 2002).

The analysis results show that the predictions from the complete truss model and the simplified truss model are almost identical. This shows that the assumptions taken to simplify the truss model are quite reasonable by keeping the method simple enough without the need to perform numerical iterations, yet maintaining its accuracy. Overall, it can be concluded that the proposed model predictions are in average closer to experimental shear strength of RC walls while still being on the conservative side and the predictions have lower

coefficient of variation values as compared to other method predictions. In addition, the truss models can predict the shear strength of RC walls with consistent accuracy for wide ranges of wall height to length ratios and concrete compressive strengths. This is not the case for the predictions from building codes (Comite European de Normalisation 2004; ACI Committee 318 2014) and other analytical models (Gupta and Rangan 1998; Hwang and Lee 2002). Their predictions show non-uniform accuracies for certain ranges of wall height to length ratios and concrete compressive strengths. Furthermore, in their formulation for calculating the shear strength of RC walls, the contribution of dowel action from reinforced boundary element is not included and hence their predictions are also less accurate for RC walls with high ratio of vertical reinforcement in boundary element.

6.2 Future Works

Another experimental study to investigate size effect on the shear behaviour of HSC walls may become point of interest in the future. In this regard, several HSC wall specimens with different scale are required. However, since the shear strength of HSC walls is quite high, thus capacity of testing equipment in the structural laboratory can be a challenge to deal with. Moreover, HSC walls with distributed loading tests may also become point of interest in the future in order to simulate different type of lateral loadings beside earthquake.

For the analytical work, the analytical models developed in this study can only be used to predict the shear strength of RC walls with predetermined reinforcement. However, for practical purposes, engineers may need a method for designing RC walls with only applied loadings as known variables. Therefore, converting the simplified truss model or the proposed strut-tie model to become design methods to be used by engineers may become point of interest in the future. Furthermore, it may become point of interest in the future to improve the complete truss model further in order to be able to predict shear force and drift relationship as well as cyclic hysteresis loops of RC walls. In addition to shear capacity, it may also become point of interest in the future to

address other important parameters, such as ductility, stiffness, and energy dissipation capacity of RC walls.

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APPENDIX A
EXPERIMENTAL DATA

In this part, experimental data that are used to plot graphs in Chapter 4 are presented. These data include forces, displacements, drifts, deformations, and strains recorded during experiment (for specimens J1 to J7).

A.1 Specimen J1

Table A.1 Forces, displacements, drifts, and deformations of specimen J1 in positive direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
0.010	0.000	0.00	0.002	0.000	0.000	0.002	-	-	-	-
277.510	0.580	0.05	0.322	0.035	0.130	0.157	55.49	6.03	22.40	27.05
480.710	1.180	0.10	0.920	0.048	0.442	0.431	77.95	4.03	37.42	36.50
635.050	2.330	0.19	2.049	0.025	1.328	0.696	87.93	1.07	56.98	29.88
784.710	3.510	0.29	3.433	0.060	2.201	1.172	97.81	1.71	62.72	33.38
892.290	4.685	0.39	4.837	0.093	3.117	1.628	103.25	1.97	66.53	34.74

Table A.2 Forces, displacements, drifts, and deformations of specimen J1 in negative direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
0.010	0.000	0.00	0.002	0.000	0.000	0.002	-	-	-	-
-303.150	-0.610	-0.05	-0.462	-0.028	-0.118	-0.316	75.68	4.51	19.40	51.77
-486.280	-1.160	-0.10	-0.874	-0.020	-0.408	-0.445	75.33	1.72	35.21	38.39
-617.400	-2.530	-0.21	-2.432	-0.080	-1.174	-1.177	96.11	3.16	46.42	46.53
-766.990	-3.610	-0.30	-3.497	-0.113	-1.774	-1.611	96.87	3.12	49.14	44.61
-840.610	-4.765	-0.40	-3.901	-0.093	-2.141	-1.667	81.87	1.94	44.94	34.99
-902.440	-5.970	-0.50	-5.512	-0.540	-2.966	-2.006	92.33	9.05	49.69	33.60
-1054.040	-7.230	-0.60	-6.709	-0.473	-3.780	-2.456	92.79	6.54	52.28	33.97
-1142.460	-8.365	-0.70	-8.038	-0.583	-4.589	-2.866	96.09	6.96	54.86	34.26
-1209.600	-9.455	-0.79	-9.273	-0.650	-5.450	-3.173	98.07	6.87	57.65	33.55
-756.810	-12.185	-1.02	-15.818	-0.998	-12.771	-2.050	129.82	8.19	104.81	16.83
-407.280	-14.530	-1.21	-21.940	-2.708	-17.346	-1.886	151.00	18.63	119.38	12.98

Table A.3 Vertical strain profile at $y = 0$ mm; $z = 50$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-2	0	1	1	0	0	-2
-0.05		-1	3	-53	-50	-115	-154	-250
-0.10		117	133	-17	-63	-167	-167	-284
-0.21		580	947	440	-5	-102	523	-329
-0.30		975	1280	646	21	-7	932	-363
-0.40		1263	1491	915	17	60	1331	-251
-0.50		1623	1770	1094	128	30	1785	-194
-0.60		2015	2103	1323	185	-1	2085	-126
-0.70		2281	2321	1480	168	-60	2036	-121
-0.79		2487	2496	1590	150	-120	1758	-62
-1.02		1260	1256	769	102	-17	2583	-89
-1.21		703	685	454	71	-1	1493	-409

Table A.4 Vertical strain profile at $y = 0$ mm; $z = 500$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-2	1	0	1	-1	1	0
-0.05		-51	-14	-81	-104	-137	-19	-184
-0.10		20	54	-53	-125	-225	81	-192
-0.21		606	317	211	-215	635	29	-7
-0.30		999	607	2759	-205	482	17	91
-0.40		1206	724	2894	-233	551	23	117
-0.50		1545	855	2479	1336	658	31	184
-0.60		1964	1096	2731	2799	881	31	276
-0.70		2253	1211	2727	3240	1555	27	373
-0.79		2491	1265	2687	3807	2204	25	469
-1.02		1501	710	1908	2661	8789	21	1373
-1.21		969	462	1375	2022	8153	43	880

Table A.5 Vertical strain profile at $y = 0$ mm; $z = 950$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-3	3	0	-1	0	-1	-1
-0.05		-85	-51	-138	-188	-193	-91	-37
-0.10		-102	-20	-124	-98	400	-118	-26
-0.21		-35	17	408	-1	50	-57	49
-0.30		309	452	364	-116	-36	-63	69
-0.40		553	825	251	-155	-97	-72	91
-0.50		960	1320	453	-263	-220	-76	122
-0.60		1358	1352	510	-325	-300	-69	153
-0.70		1668	1196	665	-350	-366	-74	165
-0.79		1995	1093	735	-368	-439	-76	179
-1.02		2893	596	178	0	-500	-62	527
-1.21		1452	532	403	0	-179	-56	329

Table A.6 Vertical strain profile at x = -462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-2	-2	-2	0
-0.05		47	-1	-34	-80
-0.10		196	117	91	42
-0.21		801	580	539	404
-0.30		1206	975	948	694
-0.40		1481	1263	1253	896
-0.50		1797	1623	1719	1232
-0.60		2269	2015	2224	1616
-0.70		2612	2281	2552	1787
-0.79		2882	2487	2787	1920
-1.02		1555	1260	1374	853
-1.21		845	703	736	400

Table A.7 Vertical strain profile at x = -462 mm; z = 450 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-2	-2	-2	-1
-0.05		-8	-51	-69	-80
-0.10		67	20	9	-7
-0.21		695	606	600	529
-0.30		1099	999	996	900
-0.40		1330	1206	1218	1129
-0.50		1676	1545	1571	1481
-0.60		2128	1964	2027	1909
-0.70		2451	2253	2341	2196
-0.79		2712	2491	2627	2425
-1.02		1537	1501	1601	1583
-1.21		907	969	1032	1002

Table A.8 Vertical strain profile at x = -462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-2	-3	-3	-3
-0.05		-64	-85	-74	-76
-0.10		-67	-102	-82	-87
-0.21		47	-35	63	125
-0.30		338	309	406	353
-0.40		486	553	589	430
-0.50		769	960	885	627
-0.60		1037	1358	1263	921
-0.70		1208	1668	1578	1124
-0.79		1375	1995	1926	1283
-1.02		-223	2893	2468	-1097
-1.21		-4647	1452	1711	-3263

Table A.9 Vertical strain profile at x = 462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		1	-2	-2	-2
-0.05		-284	-250	-275	-394
-0.10		-354	-284	-316	-496
-0.21		-449	-329	-361	-588
-0.30		-545	-363	-408	-670
-0.40		-718	-251	-367	-757
-0.50		-739	-194	-375	-817
-0.60		-838	-126	-384	-876
-0.70		-870	-121	-412	-875
-0.79		-942	-62	-329	-773
-1.02		-1117	-89	-213	-412
-1.21		-1918	-409	-306	-57

Table A.10 Vertical strain profile at x = 462 mm; z = 450 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		0	0	-1	-1
-0.05		-126	-184	-206	-211
-0.10		-139	-192	-218	-226
-0.21		26	-7	-54	-83
-0.30		45	91	17	-2
-0.40		37	117	58	36
-0.50		84	184	164	123
-0.60		124	276	255	199
-0.70		184	373	351	291
-0.79		256	469	453	383
-1.02		1203	1373	1589	1468
-1.21		681	880	932	814

Table A.11 Vertical strain profile at x = 462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		0	-1	-2	-2
-0.05		-71	-37	-73	-93
-0.10		-70	-26	-64	-83
-0.21		-9	49	8	-4
-0.30		41	69	42	17
-0.40		54	91	66	45
-0.50		88	122	100	78
-0.60		96	153	126	103
-0.70		95	165	142	118
-0.79		99	179	156	131
-1.02		373	527	435	435
-1.21		222	329	270	273

Table A.12 Horizontal strain profile at $x = -475$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-1	-1	-2	-1	1	0
-0.05		-13	17	22	35	42	29
-0.10		-34	12	3	50	77	48
-0.21		61	79	38	62	94	75
-0.30		107	199	179	230	178	89
-0.40		140	242	294	390	313	101
-0.50		225	287	396	583	563	107
-0.60		229	317	434	687	777	119
-0.70		248	345	474	776	1029	135
-0.79		263	361	507	864	1346	155
-1.02		239	247	468	1396	2694	104
-1.21		228	211	460	1302	2393	68

Table A.13 Horizontal strain profile at $x = -350$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		1	0	-1	-2	-1	0
-0.05		30	33	26	45	50	43
-0.10		42	35	105	315	262	41
-0.21		171	226	606	2162	781	264
-0.30		329	682	777	3665	1957	295
-0.40		476	1076	876	3626	2761	283
-0.50		517	1815	987	3546	3429	385
-0.60		602	1926	1087	3724	4730	428
-0.70		691	2002	1141	3795	1204	483
-0.79		772	2070	1153	3800	1354	498
-1.02		796	2066	1107	2176	1796	392
-1.21		727	1980	1040	2027	1132	-450

Table A.14 Horizontal strain profile at $x = -50$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-1	0	1	-1	-1	-2
-0.05		30	17	51	66	57	82
-0.10		63	35	62	21	4	113
-0.21		42	928	93	-27	301	290
-0.30		48	2069	1065	161	407	295
-0.40		100	2411	1659	339	334	312
-0.50		315	3153	2072	3522	193	319
-0.60		384	3239	2199	21733	240	321
-0.70		421	3270	2265	13867	293	333
-0.79		454	3293	2282	13910	312	342
-1.02		417	2408	1510	13385	0	317
-1.21		360	2087	1187	12735	0	362

Table A.15 Horizontal strain profile at $x = 350$ mm; $y = 0$ mm.

DRIFT (%)	$z =$ (mm)	0 (μ)	200 (μ)	400 (μ)	600 (μ)	800 (μ)	1000 (μ)
0.00		-2	-1	-1	-1	-1	-2
-0.05		-67	14	52	52	83	56
-0.10		-94	45	325	1255	183	78
-0.21		-48	1922	1740	1133	650	101
-0.30		-30	2923	1528	929	888	110
-0.40		-7	3613	1520	865	1054	122
-0.50		12	6233	1288	760	1467	141
-0.60		-48	6220	1276	768	1481	142
-0.70		-78	5758	1184	782	1490	146
-0.79		-83	5640	1182	784	1499	149
-1.02		379	6604	234	905	1730	172
-1.21		262	3092	1511	1160	1653	160

Table A.16 Horizontal strain profile at $x = 475$ mm; $y = 0$ mm.

DRIFT (%)	$z =$ (mm)	0 (μ)	200 (μ)	400 (μ)	600 (μ)	800 (μ)	1000 (μ)
0.00		-1	-2	0	0	0	0
-0.05		18	76	65	45	3	3
-0.10		52	101	40	68	32	16
-0.21		105	171	174	115	81	25
-0.30		167	270	243	155	160	27
-0.40		273	399	336	208	211	31
-0.50		446	717	623	356	309	38
-0.60		604	1223	873	426	306	36
-0.70		641	1644	1082	513	317	37
-0.79		694	1925	1217	587	332	36
-1.02		683	2331	1716	1302	661	44
-1.21		428	2447	974	952	630	44

A.2 Specimen J2

Table A.17 Forces, displacements, drifts, and deformations of specimen J2 in positive direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
0.060	0.000	0.00	-0.002	0.000	0.000	-0.002	-	-	-	-
228.800	0.580	0.05	0.680	0.005	0.356	0.319	117.18	0.86	61.34	54.98
349.780	1.250	0.10	1.197	0.030	0.566	0.602	95.79	2.40	45.26	48.14
558.250	2.440	0.20	2.346	0.135	1.252	0.959	96.16	5.53	51.33	39.29
677.260	3.580	0.30	3.319	0.230	1.815	1.274	92.71	6.42	50.70	35.58
834.890	4.740	0.40	4.563	0.250	2.576	1.736	96.26	5.27	54.35	36.63
974.510	6.040	0.50	5.971	0.385	3.327	2.259	98.86	6.37	55.08	37.41
1072.540	7.250	0.60	7.089	0.530	4.083	2.476	97.78	7.31	56.31	34.15
1185.710	8.430	0.70	8.300	0.580	4.817	2.903	98.46	6.88	57.14	34.43
1264.750	9.580	0.80	9.479	0.670	5.644	3.166	98.95	6.99	58.91	33.04

Table A.18 Forces, displacements, drifts, and deformations of specimen J2 in negative direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
0.060	0.000	0.00	-0.002	0.000	0.000	-0.002	-	-	-	-
-251.110	-0.590	-0.05	-0.407	0.030	-0.139	-0.298	69.01	-5.08	23.55	50.55
-504.580	-1.120	-0.09	-1.074	0.005	-0.531	-0.547	95.86	-0.45	47.44	48.86
-647.870	-2.320	-0.19	-2.302	-0.095	-1.299	-0.908	99.22	4.09	56.00	39.12
-829.160	-3.490	-0.29	-3.629	-0.150	-1.920	-1.559	103.99	4.30	55.02	44.67
-940.720	-4.540	-0.38	-4.595	-0.215	-2.481	-1.899	101.21	4.74	54.66	41.82
-1099.200	-6.150	-0.51	-6.193	-0.395	-3.455	-2.343	100.71	6.42	56.19	38.10
-1225.740	-7.210	-0.60	-7.319	-0.475	-4.097	-2.747	101.51	6.59	56.83	38.09
-1270.820	-8.120	-0.68	-8.149	-0.530	-4.668	-2.951	100.35	6.53	57.49	36.34
-911.110	-9.470	-0.79	-11.449	-0.500	-8.147	-2.802	120.89	5.28	86.03	29.59
-718.050	-10.560	-0.88	-12.927	-0.500	-9.835	-2.592	122.41	4.73	93.14	24.54

Table A.19 Vertical strain profile at $y = 0$ mm; $z = 50$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-3	0	0	1	0	-1	0
-0.05		-77	-35	0	28	-123	-132	-335
-0.09		413	99	0	-26	-104	-246	-529
-0.19		684	435	0	218	279	62	-728
-0.29		1106	889	0	269	630	268	-879
-0.38		1407	1164	0	423	818	534	-900
-0.51		1892	1533	0	532	961	1355	-862
-0.60		2329	1869	0	589	921	1647	-737
-0.68		2566	2000	0	622	788	1618	-871
-0.79		1823	1270	0	-326	587	1189	-808
-0.88		1581	1093	0	-322	601	1102	-835

Table A.20 Vertical strain profile at $y = 0$ mm; $z = 500$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-1	-55	2	-1	1	1	0
-0.05		-65	-3817	-102	-117	-82	-117	-156
-0.09		6	-4053	-97	-121	-228	-172	-209
-0.19		473	-7786	923	-229	914	-83	-22
-0.29		974	-14170	1687	-247	1195	-87	72
-0.38		1312	-11573	2060	-221	1484	-95	124
-0.51		1808	-14824	2154	171	1601	-116	106
-0.60		2195	-14080	2290	1130	1813	-157	174
-0.68		2395	-13917	2248	1519	1768	-153	222
-0.79		2114	-8357	2808	1839	3751	-29	292
-0.88		1927	-8641	2263	1586	0	-50	284

Table A.21 Vertical strain profile at $y = 0$ mm; $z = 950$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	2	0	0	-1	0	2
-0.05		0	-57	0	-170	-19	-78	-83
-0.09		0	-34	0	-153	-65	-84	-62
-0.19		0	950	0	227	17	-21	-24
-0.29		0	1417	0	325	-244	-22	12
-0.38		0	1698	0	433	319	-34	44
-0.51		0	2278	0	373	852	-46	89
-0.60		0	2733	0	488	3697	-43	129
-0.68		0	2990	0	581	2181	-48	166
-0.79		0	2878	0	504	11746	-171	147
-0.88		0	2696	0	0	96107	-164	139

Table A.22 Vertical strain profile at $x = -462$ mm; $z = 50$ mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-1	-3	-1	0
-0.05		73	-77	-83	-204
-0.09		240	413	239	98
-0.19		439	684	461	232
-0.29		958	1106	854	647
-0.38		1347	1407	1139	964
-0.51		1924	1892	1542	1409
-0.60		2393	2329	1883	1689
-0.68		2650	2566	2054	1800
-0.79		2107	1823	1415	961
-0.88		1838	1581	1214	750

Table A.23 Vertical strain profile at $x = -462$ mm; $z = 450$ mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-1	-1	-2	-1
-0.05		28	-65	-99	-132
-0.09		134	6	-4	-51
-0.19		791	473	529	589
-0.29		1229	974	1020	1108
-0.38		1494	1312	1375	1455
-0.51		1846	1808	1931	1962
-0.60		2170	2195	2400	2368
-0.68		2334	2395	2632	2563
-0.79		1834	2114	2293	2027
-0.88		1787	1927	2127	1855

Table A.24 Vertical strain profile at $x = -462$ mm; $z = 950$ mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-1	0	-1	0
-0.05		-21	0	-76	-145
-0.09		-12	0	-70	-138
-0.19		259	0	174	96
-0.29		550	0	440	288
-0.38		735	0	646	452
-0.51		1069	0	1017	830
-0.60		1342	0	1318	1126
-0.68		1476	0	1475	1324
-0.79		945	0	798	245
-0.88		758	0	702	65

Table A.25 Vertical strain profile at x = 462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		0	0	-2	-1
-0.05		0	-335	-345	-486
-0.09		0	-529	-517	-663
-0.19		0	-728	-689	-883
-0.29		0	-879	-797	-1035
-0.38		0	-900	-842	-1085
-0.51		0	-862	-931	-1202
-0.60		0	-737	-1019	-1425
-0.68		0	-871	-1346	-2188
-0.79		0	-808	0	0
-0.88		0	-835	0	0

Table A.26 Vertical strain profile at x = 462 mm; z = 450 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		9	0	0	-1
-0.05		34114	-156	0	-253
-0.09		39181	-209	0	-303
-0.19		39812	-22	0	-202
-0.29		39886	72	0	-156
-0.38		39915	124	0	-109
-0.51		-72114	106	0	-31
-0.60		-80366	174	0	41
-0.68		-37596	222	0	76
-0.79		40674	292	0	-839
-0.88		40666	284	0	0

Table A.27 Vertical strain profile at x = 462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		0	2	-2	0
-0.05		-73	-83	-104	-130
-0.09		-71	-62	-90	-129
-0.19		-22	-24	-57	-106
-0.29		24	12	-27	-82
-0.38		83	44	2	-50
-0.51		114	89	43	-16
-0.60		168	129	73	-6
-0.68		167	166	101	-3
-0.79		-94	147	72	0
-0.88		-190	139	67	0

Table A.28 Horizontal strain profile at $x = -475$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		2	1	1	1	0	1
-0.05		-1	-12	-2	20	0	25
-0.09		55	52	-26	14	0	46
-0.19		147	166	23	-65	0	149
-0.29		257	210	97	-83	0	278
-0.38		310	285	136	12	0	350
-0.51		382	417	230	224	0	475
-0.60		450	558	352	393	0	557
-0.68		451	636	428	496	0	650
-0.79		344	650	582	1277	0	544
-0.88		319	615	618	1396	0	470

Table A.29 Horizontal strain profile at $x = -350$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	0	-1	1	0
-0.05		49	43	7	50	27	0
-0.09		311	461	74	48	-3	0
-0.19		111	545	1343	1507	1709	0
-0.29		185	802	1671	2932	2484	0
-0.38		254	758	1739	3937	2813	0
-0.51		283	746	1364	5318	2985	0
-0.60		319	809	1335	5594	3216	0
-0.68		282	708	1181	5787	3362	0
-0.79		259	436	649	4283	4998	0
-0.88		262	483	715	3587	4672	0

Table A.30 Horizontal strain profile at $x = -50$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-1	0	1	1	0	0
-0.05		212	47	24	50	79	0
-0.09		133	-4	-8	37	54	0
-0.19		443	350	3058	-43	157	0
-0.29		465	83	1498	-57	539	0
-0.38		531	108	1239	-66	806	0
-0.51		576	76	817	848	1112	0
-0.60		577	40	945	1109	1264	0
-0.68		612	158	746	1094	1429	0
-0.79		553	-63	1772	0	1564	0
-0.88		546	-36	984	0	1560	0

Table A.31 Horizontal strain profile at x = 350 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	1	0	-1	0	0
-0.05		-84	-9	52	40	140	78
-0.09		-171	-20	69	64	343	103
-0.19		-270	1555	240	320	191	100
-0.29		-160	1989	341	417	201	112
-0.38		-82	2339	367	512	181	113
-0.51		125	1034	375	454	152	113
-0.60		326	474	411	483	129	124
-0.68		606	803	424	481	103	124
-0.79		625	179	324	362	103	117
-0.88		613	192	324	340	104	112

Table A.32 Horizontal strain profile at x = 475 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		2	1	1	1	1	0
-0.05		46	68	50	16	0	-12
-0.09		102	129	63	9	-9	-8
-0.19		127	195	103	43	26	26
-0.29		242	360	165	117	55	35
-0.38		341	571	250	180	105	42
-0.51		482	1020	501	268	150	52
-0.60		717	1412	774	383	234	82
-0.68		697	1622	917	470	311	100
-0.79		-124	1597	933	510	387	126
-0.88		-258	1480	874	500	391	128

A.3 Specimen J3

Table A.33 Forces, displacements, drifts, and deformations of specimen J3 in positive direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
0.010	0.000	0.00	0.002	0.000	-0.001	0.002	-	-	-	-
258.580	0.650	0.05	0.601	0.038	0.189	0.375	92.41	5.77	29.01	57.63
444.970	1.190	0.10	1.050	-0.008	0.433	0.624	88.20	-0.63	36.35	52.47
595.560	2.320	0.19	2.058	0.120	1.024	0.914	88.72	5.17	44.14	39.41
805.140	3.550	0.30	3.469	0.155	1.653	1.661	97.73	4.37	46.57	46.79
979.940	4.930	0.41	4.975	0.368	2.530	2.078	100.91	7.45	51.31	42.14
1095.480	5.940	0.50	5.739	0.315	3.081	2.343	96.62	5.30	51.87	39.45
1237.000	7.140	0.60	7.316	0.495	3.890	2.932	102.47	6.93	54.48	41.06
1322.330	8.140	0.68	8.070	0.545	4.374	3.150	99.13	6.70	53.74	38.70
1402.760	9.490	0.79	9.389	0.780	5.125	3.484	98.93	8.22	54.00	36.71

Table A.34 Forces, displacements, drifts, and deformations of specimen J3 in negative direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
0.010	0.000	0.00	0.002	0.000	-0.001	0.002	-	-	-	-
-346.910	-0.570	-0.05	-0.417	-0.005	-0.165	-0.248	73.24	0.88	28.90	43.46
-516.150	-1.140	-0.10	-1.041	-0.065	-0.393	-0.583	91.32	5.70	34.45	51.18
-684.370	-2.420	-0.20	-2.277	-0.108	-1.172	-0.998	94.10	4.44	48.43	41.23
-860.180	-3.560	-0.30	-3.494	-0.100	-1.809	-1.585	98.14	2.81	50.81	44.52
-992.980	-4.580	-0.38	-4.890	-0.160	-2.534	-2.197	106.78	3.49	55.32	47.96
-1157.920	-5.920	-0.49	-6.090	-0.123	-3.219	-2.748	102.86	2.07	54.38	46.41
-1304.680	-7.210	-0.60	-7.401	-0.250	-3.866	-3.285	102.65	3.47	53.62	45.56
-1392.020	-8.220	-0.69	-8.337	-0.135	-4.512	-3.690	101.43	1.64	54.89	44.89
-1458.850	-9.150	-0.76	-9.314	-0.010	-5.184	-4.120	101.79	0.11	56.65	45.03
-773.620	-13.650	-1.14	-12.777	-0.030	-9.577	-3.169	93.60	0.22	70.16	23.22

Table A.35 Vertical strain profile at $y = 0$ mm; $z = 50$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	0	-1	1	-1	0
-0.05		30	-8	0	-46	-161	-462	-336
-0.10		245	224	0	24	-125	-531	-445
-0.20		698	692	0	177	-100	-328	-568
-0.30		1187	1037	0	203	1350	462	-624
-0.38		1533	1425	0	195	1142	263	-678
-0.49		2000	1901	0	242	1553	115	-767
-0.60		2369	2332	0	234	1593	137	-847
-0.69		2655	2683	0	216	1450	193	-856
-0.76		3058	3067	0	174	1254	222	-743
-1.14		1860	1596	0	0	0	0	-379

Table A.36 Vertical strain profile at $y = 0$ mm; $z = 500$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	0	0	-1	0	-1	-1
-0.05		-13	32	0	-187	-195	-217	-217
-0.10		243	305	0	-211	-308	-252	-242
-0.20		958	801	0	-204	-401	-315	-141
-0.30		1563	1298	0	-25	-227	-307	-94
-0.38		2064	1617	0	1302	-289	-317	-55
-0.49		2639	2074	0	1872	-265	-345	-29
-0.60		3184	2375	0	2320	258	-372	43
-0.69		3616	478	0	2645	813	-381	118
-0.76		4003	321	0	2792	1212	-362	196
-1.14		3317	-4	0	0	0	0	201

Table A.37 Vertical strain profile at $y = 0$ mm; $z = 950$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	0	0	-1	2	-2	0
-0.05		-70	-77	0	-194	-176	-108	-125
-0.10		-73	-45	0	-199	-141	-125	-110
-0.20		199	181	0	-67	-210	-121	-97
-0.30		446	580	0	-97	-248	-113	-64
-0.38		683	807	0	-207	-288	-131	-23
-0.49		970	814	0	-243	-317	-146	34
-0.60		1287	649	0	-186	-377	-161	97
-0.69		1552	615	0	-205	-405	-174	138
-0.76		1730	459	0	-193	-400	-186	189
-1.14		-320	93	0	0	0	0	145

Table A.38 Vertical strain profile at x = -462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		0	0	0	0
-0.05		0	30	57	87
-0.10		0	245	289	375
-0.20		0	698	635	625
-0.30		0	1187	1116	983
-0.38		0	1533	1466	1209
-0.49		0	2000	1915	1421
-0.60		0	2369	2302	1681
-0.69		0	2655	2604	1826
-0.76		0	3058	3030	1954
-1.14		0	1860	1802	908

Table A.39 Vertical strain profile at x = -462 mm; z = 450 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		0	0	-1	1
-0.05		-82	-13	-5	11
-0.10		109	243	302	221
-0.20		714	958	712	642
-0.30		1252	1563	1048	918
-0.38		1697	2064	1305	1150
-0.49		2173	2639	1617	1434
-0.60		2612	3184	1888	1647
-0.69		2952	3616	2074	1882
-0.76		3242	4003	2229	2225
-1.14		2450	3317	1863	1848

Table A.40 Vertical strain profile at x = -462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		-1	0	-1	0
-0.05		-94	-70	-47	-29
-0.10		-79	-73	-23	2
-0.20		60	199	438	471
-0.30		309	446	804	817
-0.38		528	683	1121	1063
-0.49		756	970	1497	1365
-0.60		992	1287	1877	1668
-0.69		1167	1552	2199	1891
-0.76		1319	1730	2442	2054
-1.14		-446	-320	2152	1148

Table A.41 Vertical strain profile at x = 462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		1	0	1	0
-0.05		-492	-336	-352	-338
-0.10		-605	-445	-467	-446
-0.20		-745	-568	-577	-560
-0.30		-823	-624	-660	-634
-0.38		-875	-678	-732	-700
-0.49		-944	-767	-849	-807
-0.60		-1021	-847	-1024	-966
-0.69		-523	-856	-1157	-1166
-0.76		-506	-743	-1241	-1350
-1.14		-1099	-379	-2087	601

Table A.42 Vertical strain profile at x = 462 mm; z = 450 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		-1	-1	-1	1
-0.05		-152	-217	-286	-273
-0.10		-562	-242	-312	-296
-0.20		-344	-141	-157	-171
-0.30		-86573	-94	-122	-146
-0.38		-81370	-55	-87	-118
-0.49		-22666	-29	-67	-96
-0.60		-20987	43	-28	-65
-0.69		-20259	118	32	-13
-0.76		-28833	196	95	45
-1.14		-32665	201	174	0

Table A.43 Vertical strain profile at x = 462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		2	0	0	-1
-0.05		-79	-125	-89	-113
-0.10		-78	-110	-71	-99
-0.20		-39	-97	-64	-77
-0.30		40	-64	-33	-42
-0.38		90	-23	1	-3
-0.49		161	34	47	34
-0.60		228	97	95	78
-0.69		276	138	130	110
-0.76		309	189	168	176
-1.14		229	145	113	0

Table A.44 Horizontal strain profile at $x = -475$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	1	-1	-1	-1	0
-0.05		-12	-10	4	-12	3	18
-0.10		60	6	-22	6	-3	42
-0.20		154	64	48	217	28	164
-0.30		252	181	135	367	125	271
-0.38		310	252	230	484	232	343
-0.49		393	347	321	617	411	469
-0.60		458	446	426	741	583	618
-0.69		497	520	485	835	690	867
-0.76		497	638	530	903	774	1089
-1.14		400	529	717	2132	2216	4621

Table A.45 Horizontal strain profile at $x = -350$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		1	-2	1	-1	-1	1
-0.05		51	21	241	65	18	19
-0.10		144	68	770	842	45	13
-0.20		366	208	862	1487	456	151
-0.30		414	598	1305	2013	968	183
-0.38		359	725	1378	2150	1824	202
-0.49		372	878	1556	2475	2162	247
-0.60		399	872	1559	2676	2524	319
-0.69		405	661	1600	2807	2761	370
-0.76		483	748	1665	2962	3107	435
-1.14		503	698	1623	3425	0	534

Table A.46 Horizontal strain profile at $x = -50$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	-1	-1	0	0
-0.05		181	-5	1	24	63	73
-0.10		737	8	-5	-40	17	101
-0.20		864	1723	363	968	-21	509
-0.30		1183	1876	918	1686	322	685
-0.38		1426	1779	2302	2197	466	792
-0.49		-102	1983	2707	2654	537	887
-0.60		-88	2036	2747	2830	883	1053
-0.69		-75	1977	2724	3253	2590	1090
-0.76		-73	1924	2654	3411	3841	1220
-1.14		-160	1694	3029	0	0	1171

Table A.47 Horizontal strain profile at x = 350 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		2	2	-1	0	0	1
-0.05		-68	-10	21	234	18	50
-0.10		-114	-20	38	465	72	50
-0.20		-134	1914	706	1066	68	70
-0.30		-17	2508	1000	1210	82	81
-0.38		552	2683	1138	1282	85	99
-0.49		697	3023	1314	1411	91	125
-0.60		816	4346	1485	1470	97	155
-0.69		896	2567	1598	1546	106	185
-0.76		949	2618	1727	2371	235	227
-1.14		0	0	0	2146	227	222

Table A.48 Horizontal strain profile at x = 475 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		2	0	1	-1	-2	-1
-0.05		54	0	54	-1	-52	-41
-0.10		111	0	96	-17	-27	-25
-0.20		200	0	177	59	-5	-11
-0.30		399	0	278	98	29	17
-0.38		634	0	505	195	107	52
-0.49		942	0	718	304	192	103
-0.60		1317	0	928	424	290	171
-0.69		1874	0	1098	512	358	218
-0.76		2003	0	1249	588	453	291
-1.14		494	0	899	593	432	277

A.4 Specimen J4

Table A.49 Forces, displacements, drifts, and deformations of specimen J4 in positive direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
-0.150	0.000	0.00	0.001	0.000	0.005	-0.003	-	-	-	-
262.340	0.610	0.05	0.433	-0.050	-0.208	0.691	71.02	-8.20	-34.09	113.30
358.120	1.230	0.10	0.816	-0.035	-0.281	1.132	66.34	-2.85	-22.88	92.07
436.130	2.310	0.19	1.847	-0.010	0.214	1.644	79.98	-0.43	9.25	71.16
590.240	3.590	0.30	2.999	0.010	0.521	2.467	83.53	0.28	14.52	68.73
695.840	4.810	0.40	4.533	0.060	1.228	3.245	94.25	1.25	25.53	67.47
792.190	6.100	0.51	6.101	0.160	1.724	4.217	100.02	2.62	28.26	69.13
810.740	6.480	0.54	7.081	0.215	2.251	4.615	109.27	3.32	34.74	71.21
786.510	7.250	0.60	8.147	0.455	2.746	4.946	112.38	6.28	37.88	68.22
602.210	8.420	0.70	9.130	1.285	3.614	4.231	108.43	15.26	42.92	50.25

Appendix A: Experimental Data

Table A.50 Forces, displacements, drifts, and deformations of specimen J4 in negative direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
-0.150	0.000	0.00	0.001	0.000	0.005	-0.003	-	-	-	-
-212.350	-0.620	-0.05	-0.480	-0.050	-0.004	-0.426	77.50	8.06	0.72	68.71
-331.720	-1.230	-0.10	-0.927	-0.055	-0.087	-0.785	75.35	4.47	7.05	63.83
-411.710	-2.510	-0.21	-1.889	-0.100	-0.735	-1.054	75.25	3.98	29.27	41.99
-482.100	-3.550	-0.30	-2.531	-0.115	-1.028	-1.389	71.30	3.24	28.95	39.12
-551.490	-4.760	-0.40	-3.359	-0.130	-1.426	-1.804	70.58	2.73	29.95	37.90
-638.200	-5.960	-0.50	-4.192	-0.095	-1.795	-2.301	70.33	1.59	30.12	38.62
-729.380	-7.240	-0.60	-5.536	0.000	-2.524	-3.012	76.47	0.00	34.86	41.61
-816.210	-8.400	-0.70	-8.295	0.065	-4.298	-4.062	98.75	-0.77	51.16	48.36
-826.120	-8.760	-0.73	-8.902	-0.240	-4.412	-4.250	101.62	2.74	50.37	48.51
-817.810	-9.760	-0.81	-11.128	-0.755	-5.517	-4.856	114.02	7.74	56.53	49.76
-805.690	-10.850	-0.90	-12.492	-1.700	-5.772	-5.019	115.13	15.67	53.20	46.26
-655.350	-12.040	-1.00	-14.124	-3.045	-6.153	-4.927	117.31	25.29	51.10	40.92
-496.250	-13.290	-1.11	-16.886	-4.200	-8.282	-4.404	127.06	31.60	62.32	33.14

Appendix A: Experimental Data

Table A.51 Vertical strain profile at $y = 0$ mm; $z = 50$ mm.

DRIFT (%)	x = (mm)	-462 (μ)	-400 (μ)	-258 (μ)	-200 (μ)	0 (μ)	200 (μ)	258 (μ)	400 (μ)	462 (μ)
0.00		1	-1	0	0	-2	-1	0	-2	-1
0.05		-635	-421	-245	-383	-237	-6	18	89	133
0.10		-837	-545	-255	-368	229	42	223	382	510
0.19		-1149	-593	-162	-222	760	70	350	635	825
0.30		-1574	-700	49	-265	1566	134	641	1098	1422
0.40		-1924	-657	837	-450	1798	153	862	1502	1930
0.51		-2357	-820	1213	-603	1998	173	1081	1910	2401
0.54		-2726	-1027	1319	-671	1916	163	1144	2088	2576
0.60		-3479	-1594	1276	-743	1659	147	1145	2149	2648
0.70		-4348	-2595	1253	-741	724	78	871	1723	2144

Table A.52 Vertical strain profile at $y = 0$ mm; $z = 450$ mm.

DRIFT (%)	x = (mm)	-462 (μ)	-400 (μ)	-258 (μ)	-200 (μ)	0 (μ)	200 (μ)	258 (μ)	400 (μ)	462 (μ)
0.00		0	-1	0	0	0	-2	0	-1	-1
0.05		-287	-293	-196	-212	0	-49	1	93	119
0.10		-348	-317	-227	-237	0	-21	192	341	311
0.19		-300	-312	-251	-302	0	-28	481	766	801
0.30		-291	-345	-313	-395	0	-7	826	1283	1355
0.40		-268	-364	-379	-518	0	117	1153	1696	1813
0.51		-248	-371	-427	-640	0	419	1441	2080	2245
0.54		-219	-357	-430	-671	0	647	1551	2212	2399
0.60		-199	-347	-375	-632	0	608	1509	2273	2497
0.70		-126	-281	-275	-570	0	647	984	1932	2266

Table A.53 Vertical strain profile at $y = 0$ mm; $z = 950$ mm.

DRIFT (%)	x = (mm)	-462 (μ)	-400 (μ)	-258 (μ)	-200 (μ)	0 (μ)	200 (μ)	258 (μ)	400 (μ)	462 (μ)
0.00		-1	0	-1	-2	0	0	0	-2	-1
0.05		-142	-183	-164	-68	-146	0	-50	-10	-29
0.10		-166	-179	-178	-75	-158	0	-36	-21	-50
0.19		-92	-138	-201	-91	-286	0	436	430	26
0.30		-13	-112	-213	-85	-362	0	1031	934	324
0.40		37	-75	-240	-109	-433	0	1495	1262	490
0.51		59	-42	-252	-123	-483	0	1899	1500	655
0.54		81	-17	-246	-117	-491	0	2032	1570	700
0.60		50	-44	-240	-108	-455	0	2185	1553	625
0.70		64	-85	-178	-105	-368	0	2167	1321	-35

Table A.54 Horizontal strain profile at x = -475 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		1	-1	0	0	-1	0
0.05		30	27	3	17	14	-2
0.10		27	39	6	24	17	-3
0.19		20	41	8	14	10	-3
0.30		2	39	10	29	21	-12
0.40		17	26	8	30	23	-12
0.51		131	27	7	32	33	0
0.54		220	42	5	34	45	7
0.60		387	223	2	34	45	7
0.70		277	1342	8	42	62	-7

Table A.55 Horizontal strain profile at x = -350 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	0	-1	-2	-2
0.05		-103	36	36	45	45	18
0.10		-132	49	42	56	51	25
0.19		-165	74	27	47	41	26
0.30		-137	226	32	108	58	55
0.40		63	650	35	127	126	71
0.51		211	1242	48	151	208	96
0.54		249	1476	53	183	291	118
0.60		142	1919	78	185	287	117
0.70		263	3915	97	250	484	147

Table A.56 Horizontal strain profile at x = -50 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-2	0	0	-1	-2	0
0.05		-1	-21	73	8	15	0
0.10		11	-27	68	21	25	0
0.19		343	17	2342	113	2	0
0.30		807	-61	8558	925	82	0
0.40		1003	-79	27825	1241	155	0
0.51		1081	-54	37891	1572	234	0
0.54		1117	28	0	1833	310	0
0.60		1083	26	0	1846	310	0
0.70		863	109	0	2041	421	0

Table A.57 Horizontal strain profile at $x = 350$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	-2	-1	-2	-1
0.05		17	10	10	-4	-1	-14
0.10		72	39	49	-22	-6	-24
0.19		92	61	91	25	572	74
0.30		124	213	116	150	1031	189
0.40		158	291	137	318	1578	266
0.51		200	346	162	594	2321	345
0.54		227	397	178	825	2811	378
0.60		242	399	181	903	3839	421
0.70		262	506	148	1590	8199	419

Table A.58 Horizontal strain profile at $x = 475$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	0	0	0	1	0
0.05		-36	0	0	-1	-1	0
0.10		-193	0	0	-11	2	0
0.19		-202	0	0	-24	87	0
0.30		-299	0	0	-16	331	0
0.40		-336	0	0	-20	609	0
0.51		-317	0	0	8	909	0
0.54		-335	0	0	27	1087	0
0.60		-324	0	0	44	1394	0
0.70		-274	0	0	17	4888	0

A.5 Specimen J5

Table A.59 Forces, displacements, drifts, and deformations of specimen J5 in positive direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
0.020	0.000	0.00	-0.004	0.000	0.000	-0.004	-	-	-	-
180.240	1.300	0.06	0.946	0.008	0.020	0.919	72.77	0.58	1.53	70.67
275.080	2.320	0.11	1.586	0.020	0.091	1.475	68.37	0.86	3.91	63.60
374.940	4.490	0.20	3.445	0.008	0.699	2.739	76.73	0.17	15.57	60.99
429.340	6.720	0.31	4.436	0.128	0.746	3.562	66.01	1.90	11.11	53.01
488.080	9.040	0.41	7.021	0.050	2.269	4.702	77.67	0.55	25.10	52.01
532.340	10.960	0.50	8.967	0.138	3.247	5.582	81.81	1.25	29.63	50.93
572.520	13.300	0.60	11.526	0.135	4.973	6.418	86.66	1.02	37.39	48.25
595.760	15.350	0.70	13.560	0.103	6.550	6.907	88.34	0.67	42.67	45.00
585.100	17.710	0.81	16.437	0.023	9.461	6.953	92.81	0.13	53.42	39.26
569.130	19.790	0.90	19.360	0.295	11.979	7.086	97.83	1.49	60.53	35.81
510.300	20.950	0.95	21.531	0.580	13.770	7.181	102.77	2.77	65.73	34.28
302.980	21.920	1.00	22.913	0.475	16.123	6.315	104.53	2.17	73.55	28.81
180.470	26.540	1.21	27.537	0.605	20.337	6.595	103.76	2.28	76.63	24.85

Table A.60 Forces, displacements, drifts, and deformations of specimen J5 in negative direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
0.020	0.000	0.00	-0.004	0.000	0.000	-0.004	-	-	-	-
-166.490	-1.370	-0.06	-0.806	-0.028	-0.305	-0.474	58.86	2.01	22.26	34.60
-281.680	-2.440	-0.11	-1.587	-0.068	-0.465	-1.054	65.03	2.77	19.06	43.21
-382.940	-4.600	-0.21	-3.485	-0.170	-0.913	-2.401	75.76	3.70	19.86	52.20
-429.120	-6.610	-0.30	-4.234	-0.005	-1.125	-3.104	64.05	0.08	17.02	46.96
-482.880	-8.800	-0.40	-6.625	-0.128	-2.405	-4.093	75.29	1.45	27.33	46.51
-521.580	-10.920	-0.50	-9.034	-0.225	-3.848	-4.961	82.73	2.06	35.24	45.43
-551.670	-13.260	-0.60	-11.671	-0.138	-5.762	-5.772	88.02	1.04	43.45	43.53
-556.970	-15.420	-0.70	-14.503	-0.105	-8.000	-6.398	94.05	0.68	51.88	41.49
-554.290	-17.780	-0.81	-18.341	-0.323	-11.511	-6.507	103.16	1.81	64.74	36.60
-524.510	-19.950	-0.91	-21.667	-0.070	-14.577	-7.020	108.61	0.35	73.07	35.19
-438.930	-22.380	-1.02	-25.642	-0.070	-19.610	-5.961	114.57	0.31	87.63	26.64
-326.500	-26.190	-1.19	-34.602	-0.015	-28.511	-6.076	132.12	0.06	108.86	23.20
-330.960	-31.090	-1.41	-41.049	-0.400	-33.530	-7.119	132.03	1.29	107.85	22.90
-342.350	-35.000	-1.59	-46.722	-1.310	-36.973	-8.439	133.49	3.74	105.64	24.11

Table A.61 Vertical strain profile at $y = 0$ mm; $z = 50$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		1	-2	-1	0	0	-2	-1
0.06		-259	-231	-161	36	0	99	58
0.11		-355	-270	-153	282	0	646	525
0.20		-506	-290	24	1843	0	914	1013
0.31		-597	-218	20	1592	0	1083	1171
0.41		-675	-116	17	1769	0	1433	1411
0.50		-713	-6	50	2109	0	1742	1611
0.60		-746	201	105	2368	0	2048	1807
0.70		-742	593	160	2543	0	2284	1933
0.81		-670	2042	102	1917	0	2477	1702
0.90		-685	2896	9	1448	0	2653	1639
0.95		-933	2897	-81	1055	0	2436	1659
1.00		-952	1097	-119	736	0	1911	984
1.21		-512	1332	35	1369	0	1418	713

Table A.62 Vertical strain profile at $y = 0$ mm; $z = 950$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-2	-1	-3	1	-1	-3
0.06		-170	-169	-163	-90	-67	-41	-31
0.11		-217	-193	-179	-82	-32	3	21
0.20		-226	-203	-198	-96	-48	493	209
0.31		-168	-227	-276	-195	1529	742	609
0.41		-117	-235	-371	2079	1811	973	929
0.50		-87	-235	-438	4404	1787	1131	1173
0.60		-52	-242	-483	3988	1415	1288	1467
0.70		-6	-238	-565	4179	827	1382	1675
0.81		92	-230	-740	4162	-175	1315	1790
0.90		166	-241	-847	3859	-519	1327	1896
0.95		231	-223	-817	3868	-780	1259	1808
1.00		141	-154	-393	3827	-1285	742	1313
1.21		137	-75	-259	3764	0	431	918

Table A.63 Vertical strain profile at y = 0 mm; z = 1950 mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-3	-2	-3	-1	-2	-2
0.06		0	-143	-197	-135	-100	-65	-73
0.11		0	-137	-203	-128	-82	-48	-63
0.20		0	-124	-202	-123	-61	-30	-61
0.31		0	-119	-247	-191	-130	-44	-64
0.41		0	-98	-262	-235	-186	-51	72
0.50		0	-14	-133	-222	-198	80	150
0.60		0	95	450	-151	-217	258	232
0.70		0	143	1044	-109	-228	550	313
0.81		0	176	1250	-110	-241	1138	383
0.90		0	194	1761	10	-236	1240	418
0.95		0	156	2479	0	0	1270	349
1.00		0	176	2834	0	0	0	-173
1.21		0	197	2607	0	0	0	-540

Table A.64 Vertical strain profile at x = -462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		0	1	-1	2
0.06		-343	-259	-396	-720
0.11		-470	-355	-530	-942
0.20		-619	-506	-726	-1207
0.31		-721	-597	-889	-1402
0.41		-826	-675	-1053	-1577
0.50		-909	-713	-1176	-1713
0.60		-993	-746	-1297	-1841
0.70		-1044	-742	-1376	-1933
0.81		-1190	-670	-1406	-2031
0.90		-1349	-685	-1361	-1977
0.95		-1380	-933	-1858	-2685
1.00		-1817	-952	-1424	-5630
1.21		-2061	-512	-678	-5671

Table A.65 Vertical strain profile at x = -462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-1	0	-1	0
0.06		-201	-170	-157	-158
0.11		-241	-217	-194	-199
0.20		-250	-226	-208	-218
0.31		-221	-168	-141	-140
0.41		-173	-117	-94	-106
0.50		-150	-87	-71	-86
0.60		-147	-52	-40	-60
0.70		-127	-6	9	-18
0.81		-37	92	102	73
0.90		14	166	181	151
0.95		42	231	248	225
1.00		18	141	164	131
1.21		0	137	161	148

Table A.66 Vertical strain profile at x = -462 mm; z = 1950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-1	0	-1	-1
0.06		-127	0	-82	-123
0.11		-125	0	-86	-143
0.20		-109	0	-83	-153
0.31		-96	0	-79	-153
0.41		-103	0	-76	-152
0.50		-135	0	-79	-184
0.60		-156	0	-109	-218
0.70		-140	0	-117	-233
0.81		-145	0	-131	-250
0.90		-122	0	-129	-258
0.95		-82	0	-62	-254
1.00		-67	0	-53	-240
1.21		-67	0	15	-116

Table A.67 Vertical strain profile at x = 462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-3	-1	0	-1
0.06		226	58	13	-45
0.11		614	525	204	96
0.20		967	1013	576	351
0.31		1249	1171	706	437
0.41		1623	1411	892	582
0.50		1890	1611	1029	725
0.60		2086	1807	1167	879
0.70		2191	1933	1262	980
0.81		1998	1702	1118	868
0.90		1877	1639	1073	801
0.95		1658	1659	978	721
1.00		976	984	500	360
1.21		504	713	309	198

Table A.68 Vertical strain profile at x = 462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-2	-3	-2	-2
0.06		-5	-31	-41	-62
0.11		26	21	9	6
0.20		610	209	205	225
0.31		975	609	623	765
0.41		1356	929	932	1096
0.50		1631	1173	1175	1348
0.60		1894	1467	1495	1648
0.70		2086	1675	1703	1878
0.81		2162	1790	1821	2013
0.90		2281	1896	1924	2120
0.95		2225	1808	1813	1967
1.00		1648	1313	1290	1337
1.21		1208	918	888	871

Table A.69 Vertical strain profile at x = 462 mm; z = 1950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-3	-2	-2	0
0.06		-61	-73	-40	-93
0.11		-58	-63	-33	-81
0.20		-66	-61	-28	-64
0.31		-63	-64	-23	-35
0.41		69	72	67	200
0.50		179	150	130	304
0.60		317	232	201	404
0.70		462	313	257	513
0.81		664	383	312	611
0.90		834	418	353	683
0.95		834	349	314	596
1.00		-549	-173	-70	-845
1.21		-737	-540	-388	-2545

Table A.70 Horizontal strain profile at x = -475 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-2	-3	0	-1	-1	-2	-2	-2	-2	0	0
0.06		60	74	49	58	52	47	34	39	48	0	24
0.11		117	125	85	85	79	71	58	61	68	0	40
0.20		222	213	129	103	89	82	68	70	76	0	51
0.31		296	338	218	136	134	96	70	59	75	0	69
0.41		383	410	296	154	145	104	76	76	101	0	84
0.50		448	473	350	173	148	108	102	121	161	0	124
0.60		523	566	467	210	148	109	111	143	225	0	148
0.70		576	693	634	252	145	107	120	170	297	0	163
0.81		608	1209	1031	414	133	92	124	263	517	0	183
0.90		847	1458	1183	578	141	70	129	450	934	0	190
0.95		1144	1490	1304	613	146	58	157	767	1703	0	183
1.00		0	1229	935	393	132	65	168	754	1674	0	183
1.21		0	1061	700	312	138	63	174	716	1517	0	178

Table A.71 Horizontal strain profile at x = -350 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-2	-1	-1	1	0	-1	1	1	-1	1	-1
0.06		2	97	39	57	25	48	27	45	31	18	45
0.11		-15	151	55	74	34	59	32	52	36	21	49
0.20		-78	169	46	86	35	66	32	60	39	23	49
0.31		359	164	204	306	97	410	182	74	37	20	51
0.41		749	1111	2148	327	246	839	524	1149	462	73	54
0.50		919	1700	3197	332	327	1017	621	766	1268	729	118
0.60		1106	2455	3608	329	382	1205	661	769	1286	1336	162
0.70		1217	3939	3732	340	421	1482	786	805	1315	1400	204
0.81		1400	1865	3474	360	454	1733	862	866	1367	1205	274
0.90		1348	1417	881	1771	538	2189	1056	906	1550	965	400
0.95		1155	1473	748	2178	680	2558	1185	206	1579	814	525
1.00		692	0	747	1604	638	2530	1184	254	1486	750	510
1.21		764	150	758	1401	570	1535	760	664	1444	509	458

Table A.72 Horizontal strain profile at $x = -50$ mm; $y = 0$ mm.

DRIFT (%)	$z =$ (mm)	0	400	800	1200	1600	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-2	-1	0	1	0	0
0.06		83	27	22	15	30	-7
0.11		218	19	24	17	36	-11
0.20		662	-26	5	15	45	-17
0.31		650	2410	67	17	46	-21
0.41		648	2639	1490	78	60	-25
0.50		672	3230	1412	68	57	-4
0.60		702	3510	1649	241	26	77
0.70		715	3950	1919	435	114	256
0.81		675	0	217	545	147	336
0.90		644	0	0	547	52	450
0.95		628	0	0	52	49	557
1.00		605	0	0	150	23	549
1.21		676	0	0	-102	-16	432

Table A.73 Horizontal strain profile at x = 350 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-1	-2	-2	0	0	-1	0	0	-2	-1	0
0.06		82	24	13	1	6	19	7	15	19	1	-12
0.11		192	84	44	4	-4	37	4	17	15	-4	-24
0.20		243	461	949	1578	1634	683	77	44	15	-12	-45
0.31		260	583	872	1370	1770	3524	2363	2227	1292	107	-61
0.41		338	959	1103	1248	1877	3588	3529	2633	5916	1390	-48
0.50		390	1817	1128	1100	1976	3770	3205	2734	2472	1870	-26
0.60		423	1583	442	1105	1934	3679	2984	2802	2402	2520	-20
0.70		466	1570	-14	955	1928	3586	2793	1651	2378	3895	-4
0.81		500	2450	-271	519	1447	2539	2056	1183	2618	2604	231
0.90		664	2545	43	503	1827	3130	2098	908	2464	1333	350
0.95		1143	2531	306	567	2722	4060	1422	911	2003	983	451
1.00		1002	2452	274	659	3010	4199	1789	947	1714	2875	422
1.21		840	2296	225	898	3113	3776	1369	749	1333	0	254

Table A.74 Horizontal strain profile at x = 475 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-2	-2	-1	-2	-2	-2	-3	-1	-2	-2	-3
0.06		-45	5	7	13	15	17	20	20	28	67	33
0.11		-18	48	23	19	27	27	31	34	46	89	57
0.20		34	108	105	146	-12	-38	4	32	57	110	76
0.31		50	145	133	150	19	-28	42	24	25	85	100
0.41		60	215	191	179	58	45	98	220	73	229	127
0.50		74	263	248	203	102	83	117	310	229	380	147
0.60		90	335	351	262	143	123	158	383	392	569	172
0.70		117	417	433	324	177	160	202	433	524	671	183
0.81		152	519	541	394	193	199	261	471	666	938	225
0.90		207	757	658	440	200	199	290	503	770	1341	262
0.95		420	1027	756	473	188	157	286	477	993	1909	272
1.00		466	1231	840	417	158	119	257	524	1678	3107	210
1.21		527	1453	973	377	137	108	258	474	1417	3217	172

A.6 Specimen J6

Table A.75 Forces, displacements, drifts, and deformations of specimen J6 in positive direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
-0.020	0.000	0.00	0.006	0.000	-0.005	0.010	-	-	-	-
257.950	1.160	0.05	1.636	0.005	0.121	1.510	141.03	0.43	10.41	130.19
347.750	2.500	0.11	2.440	0.035	0.167	2.237	97.59	1.40	6.70	89.49
434.490	4.480	0.20	4.276	-0.035	0.761	3.550	95.45	-0.78	17.00	79.23
510.740	6.610	0.30	5.704	0.000	0.718	4.986	86.30	0.00	10.86	75.44
541.820	8.620	0.39	7.613	0.293	1.805	5.516	88.32	3.39	20.94	63.99
634.250	10.940	0.50	10.570	0.220	3.109	7.240	96.61	2.01	28.42	66.18
665.190	13.140	0.60	12.983	0.195	4.645	8.142	98.80	1.48	35.35	61.97
703.510	15.390	0.70	15.331	0.208	6.177	8.947	99.62	1.35	40.13	58.14
724.140	17.510	0.80	17.529	-0.008	7.948	9.588	100.11	-0.04	45.39	54.76
575.450	20.000	0.91	19.498	0.130	11.289	8.079	97.49	0.65	56.45	40.39
346.930	21.640	0.98	23.053	0.035	15.066	7.952	106.53	0.16	69.62	36.75

Table A.76 Forces, displacements, drifts, and deformations of specimen J6 in negative direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
-0.020	0.000	0.00	0.006	0.000	-0.005	0.010	-	-	-	-
-207.780	-1.190	-0.05	-0.761	0.063	0.070	-0.894	63.97	-5.25	-5.92	75.14
-321.520	-2.160	-0.10	-1.428	0.073	0.010	-1.511	66.11	-3.36	-0.48	69.95
-419.730	-4.320	-0.20	-3.311	0.085	-0.378	-3.018	76.64	-1.97	8.74	69.87
-493.160	-6.490	-0.30	-5.243	0.048	-1.076	-4.214	80.78	-0.73	16.58	64.93
-580.450	-8.820	-0.40	-7.617	0.023	-2.010	-5.630	86.37	-0.26	22.79	63.83
-626.690	-10.840	-0.49	-9.936	0.060	-3.489	-6.507	91.66	-0.55	32.19	60.03
-647.030	-13.130	-0.60	-12.711	0.145	-5.685	-7.171	96.81	-1.10	43.30	54.62
-673.000	-15.510	-0.71	-15.624	0.200	-7.790	-8.034	100.73	-1.29	50.22	51.80
-531.360	-17.640	-0.80	-18.453	-0.415	-10.315	-7.723	104.61	2.35	58.48	43.78
-299.500	-19.430	-0.88	-21.446	-0.515	-14.327	-6.604	110.38	2.65	73.74	33.99
-253.700	-21.830	-0.99	-21.663	-0.660	-13.929	-7.074	99.24	3.02	63.81	32.40

Table A.77 Vertical strain profile at $y = 0$ mm; $z = 50$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	-1	0	-2	0	0
0.05		-331	-490	-195	-57	153	494	376
0.11		-471	-584	-120	180	1283	874	710
0.20		-652	-605	133	1469	1368	1031	913
0.30		-850	-527	410	1572	1578	1169	1064
0.39		-930	-376	561	1695	1742	1344	1195
0.50		-1106	-189	631	2117	2062	1662	1502
0.60		-1166	17	581	2095	2022	1891	1560
0.70		-1224	381	590	2281	2083	2085	1497
0.80		-1264	775	606	2305	1886	2259	1354
0.91		-1617	1743	483	1133	788	1837	929
0.98		-2149	973	280	587	462	1015	789

Table A.78 Vertical strain profile at $y = 0$ mm; $z = 950$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	0	1	-1	0	-1
0.05		-202	-173	0	-75	-34	3	21
0.11		-239	-203	0	-81	-25	33	55
0.20		-252	-220	0	-99	-68	903	774
0.30		-203	-190	0	-167	1088	1186	1108
0.39		-166	-205	0	1314	1364	1304	1307
0.50		-171	-238	0	2415	1451	1610	1631
0.60		-113	-272	0	1674	1392	1748	1861
0.70		-76	-288	0	691	1265	1877	2085
0.80		-46	-303	0	39	1150	1967	2253
0.91		155	-275	0	0	379	1536	1990
0.98		156	-178	0	0	-384	1034	1646

Table A.79 Vertical strain profile at $y = 0$ mm; $z = 1950$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	0	-1	-1	-1	0
0.05		-58	-87	-135	-141	-120	-58	-29
0.11		-57	-92	-135	-141	-127	-51	-30
0.20		-52	-92	-135	-125	-135	-48	-34
0.30		-41	-95	-174	852	-242	-59	-30
0.39		-50	-72	-166	1157	-298	-44	9
0.50		-93	-70	-26	1165	-346	205	90
0.60		-129	-34	65	1428	-236	487	148
0.70		-138	33	134	1458	-265	700	205
0.80		-146	63	195	1539	-279	1155	263
0.91		-115	107	602	1740	-240	1693	37
0.98		-73	65	616	1854	0	1813	-284

Table A.80 Vertical strain profile at x = -462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		0	0	-1	-1
0.05		-468	-331	-372	-504
0.11		-611	-471	-520	-686
0.20		-773	-652	-714	-889
0.30		-961	-850	-907	-1081
0.39		-1053	-930	-1000	-1158
0.50		-1250	-1106	-1165	-1317
0.60		-1336	-1166	-1243	-1377
0.70		-1428	-1224	-1328	-1461
0.80		-1495	-1264	-1375	-1505
0.91		-1746	-1617	-1578	-1682
0.98		-11511	-2149	-2289	-2621

Table A.81 Vertical strain profile at x = -462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-1	0	-2	-1
0.05		-207	-202	-185	-153
0.11		-243	-239	-220	-184
0.20		-266	-252	-234	-195
0.30		-243	-203	-176	-132
0.39		-226	-166	-144	-97
0.50		-234	-171	-150	-102
0.60		-198	-113	-103	-59
0.70		-178	-76	-74	-37
0.80		-159	-46	-21	-19
0.91		-7	155	150	116
0.98		57	156	133	95

Table A.82 Vertical strain profile at x = -462 mm; z = 1950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		0	0	0	-2
0.05		-77	-58	-56	-71
0.11		-83	-57	-57	-73
0.20		-88	-52	-55	-68
0.30		-85	-41	-43	-53
0.39		-92	-50	-49	-57
0.50		-120	-93	-74	-74
0.60		-133	-129	-84	-67
0.70		-130	-138	-61	-35
0.80		-114	-146	-31	-7
0.91		-107	-115	-8	-36
0.98		-141	-73	6	-194

Table A.83 Vertical strain profile at x = 462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		1	0	-1	0
0.05		487	376	418	523
0.11		775	710	620	786
0.20		1094	913	813	1094
0.30		1274	1064	952	1342
0.39		1390	1195	956	1472
0.50		1745	1502	1077	1852
0.60		1860	1560	1007	1967
0.70		2021	1497	912	2112
0.80		2110	1354	872	2173
0.91		1352	929	515	1516
0.98		694	789	447	439

Table A.84 Vertical strain profile at x = 462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		1	-1	-1	0
0.05		12	21	19	27
0.11		48	55	51	65
0.20		817	774	630	773
0.30		1113	1108	940	1083
0.39		1310	1307	1118	1304
0.50		1693	1631	1403	1677
0.60		1928	1861	1577	1933
0.70		2166	2085	1754	2177
0.80		2341	2253	1854	2342
0.91		2022	1990	1626	2105
0.98		1614	1646	1336	1726

Table A.85 Vertical strain profile at x = 462 mm; z = 1950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		0	0	-1	-2
0.05		-64	-29	-31	-30
0.11		-66	-30	-31	-31
0.20		-65	-34	-33	-34
0.30		-17	-30	-14	15
0.39		162	9	41	198
0.50		378	90	110	395
0.60		518	148	179	552
0.70		676	205	262	766
0.80		882	263	373	953
0.91		637	37	142	638
0.98		105	-284	-296	70

Table A.86 Horizontal strain profile at $x = -475$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	0	-1	-1	-1	-1	-2	-1	0	-1	-1
0.05		71	62	43	37	24	35	24	28	14	3	2
0.11		126	110	59	45	29	42	29	33	16	3	2
0.20		212	217	88	51	30	45	31	36	16	0	1
0.30		294	316	130	70	50	51	14	31	15	0	3
0.39		355	332	157	71	48	47	29	61	8	1	10
0.50		430	397	213	81	50	41	48	87	55	42	21
0.60		479	498	268	68	50	38	54	116	107	95	26
0.70		536	601	338	82	53	36	61	154	183	120	29
0.80		585	736	422	94	57	36	72	212	284	148	40
0.91		871	1380	783	203	56	-5	75	383	1081	496	40
0.98		783	1472	615	150	44	25	103	446	910	856	38

Table A.87 Horizontal strain profile at x = -350 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	-1	0	0	0	-1	0	0	-1	-1
0.05		-215	59	41	49	28	24	27	0	40	58	23
0.11		-371	36	35	54	32	30	32	0	44	69	24
0.20		-457	-3	47	57	32	30	33	0	50	81	27
0.30		67	18	194	240	474	320	53	0	46	81	27
0.39		274	754	892	273	741	779	785	0	1163	663	105
0.50		450	2392	1090	274	726	799	1350	0	2850	2284	225
0.60		577	7799	7899	566	5119	1291	2117	0	2829	3572	237
0.70		639	19981	12482	760	7907	1312	2258	0	2805	4159	321
0.80		711	28700	10941	824	9959	1384	2515	0	2860	4461	381
0.91		935	22571	7640	805	8855	1575	2796	0	2735	9763	399
0.98		610	20616	7396	730	8098	1908	2782	0	2965	8719	338

Table A.88 Horizontal strain profile at $x = -50$ mm; $y = 0$ mm.

DRIFT (%)	$z =$ (mm)	0	400	800	1200	1600	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	-2	0	-1	-1
0.05		-6	20	16	20	50	43
0.11		79	-16	16	23	53	44
0.20		271	11	3	22	69	46
0.30		153	3731	19	23	58	49
0.39		-33	5815	14079	16	52	55
0.50		-30	5165	9686	6	35	76
0.60		-75	4851	9537	23	287	814
0.70		-83	4850	9543	65	676	996
0.80		-106	4956	9515	0	1517	1155
0.91		36	434	9245	0	3124	1073
0.98		232	0	8861	0	0	914

Table A.89 Horizontal strain profile at x = 350 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	-1	-2	-1	-2	0	-1	-1	1	-1
0.05		371	-12	2	-12	-3	-4	6	20	11	7	-16
0.11		222	1034	587	7	-5	-9	4	-337	9	3	-25
0.20		51	848	1253	637	2370	993	35	-303	13	0	-33
0.30		-48	825	1151	654	2369	2097	3153	3248	2591	558	-25
0.39		-1	1030	1060	668	2093	2009	2995	11574	9333	1642	2
0.50		131	739	1004	749	2333	2267	3250	12068	13475	2841	51
0.60		312	998	938	619	2360	2275	3304	12090	7535	3684	282
0.70		450	559	308	515	2285	2173	3379	12203	7040	9696	388
0.80		678	864	-50	469	2284	2135	3414	12053	6860	17633	448
0.91		1243	1353	-833	346	2228	1790	2540	10490	6791	0	550
0.98		1374	0	272	296	3603	3204	3371	10717	6207	0	569

Table A.90 Horizontal strain profile at x = 475 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	-1	-1	-1	0	-1	-1	0	-2	-1
0.05		-64	-7	4	-6	-4	0	1	9	15	21	41
0.11		0	-41	62	-8	-7	0	-1	8	17	29	55
0.20		17	-46	130	-16	-142	0	-19	5	18	38	71
0.30		25	-41	157	7	-134	0	97	49	-28	20	104
0.39		33	-17	189	7	-97	0	104	100	5	-38	126
0.50		58	17	208	11	-77	0	114	162	38	205	162
0.60		97	75	221	14	-58	0	122	205	101	445	250
0.70		107	122	264	14	-32	0	141	227	174	648	296
0.80		109	197	329	19	4	0	156	244	281	904	327
0.91		103	1117	642	125	34	0	140	316	643	1814	316
0.98		356	1466	579	154	15	0	96	308	993	2611	185

A.7 Specimen J7

Table A.91 Forces, displacements, drifts, and deformations of specimen J7 in positive direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
-0.080	0.000	0.00	0.002	0.000	0.000	0.002	-	-	-	-
267.010	1.170	0.05	1.214	0.005	0.163	1.046	103.80	0.43	13.93	89.44
346.150	2.110	0.10	2.310	0.055	0.298	1.956	109.46	2.61	14.14	92.71
431.680	4.730	0.22	4.635	0.053	1.145	3.438	97.98	1.11	24.20	72.68
507.140	6.680	0.30	6.370	0.045	2.134	4.191	95.36	0.67	31.95	62.74
598.600	8.710	0.40	8.868	0.145	3.024	5.699	101.82	1.66	34.72	65.43
683.060	10.980	0.50	10.986	0.015	3.967	7.005	100.06	0.14	36.13	63.79
742.660	13.040	0.59	12.877	0.020	4.767	8.090	98.75	0.15	36.56	62.04
795.070	15.380	0.70	15.123	0.005	6.046	9.072	98.33	0.03	39.31	58.98
839.580	17.550	0.80	17.715	0.160	7.452	10.103	100.94	0.91	42.46	57.57
863.850	19.590	0.89	19.754	0.345	8.726	10.682	100.84	1.76	44.54	54.53
876.580	22.000	1.00	22.767	0.370	10.891	11.507	103.49	1.68	49.50	52.30
894.770	25.760	1.17	27.431	0.470	14.397	12.564	106.49	1.82	55.89	48.77
891.670	26.370	1.20	28.200	0.470	14.968	12.761	106.94	1.78	56.76	48.39
718.420	27.670	1.26	34.213	0.340	22.847	11.026	123.65	1.23	82.57	39.85
651.520	30.670	1.39	39.715	0.335	28.317	11.063	129.49	1.09	92.33	36.07
494.500	35.910	1.63	49.478	0.210	39.638	9.631	137.78	0.58	110.38	26.82

Table A.92 Forces, displacements, drifts, and deformations of specimen J7 in negative direction.

FORCE (kN)	DISP (mm)	DRIFT (%)	Δ (mm)	Δ_{SL} (mm)	Δ_S (mm)	Δ_F (mm)	Δ ratio (%)	Δ_{SL} ratio (%)	Δ_S ratio (%)	Δ_F ratio (%)
-0.080	0.000	0.00	0.002	0.000	0.000	0.002	-	-	-	-
-260.910	-1.120	-0.05	-0.681	-0.063	0.050	-0.668	60.77	5.58	-4.49	59.67
-344.110	-2.250	-0.10	-1.577	-0.080	-0.044	-1.453	70.09	3.56	1.96	64.57
-437.350	-4.390	-0.20	-4.353	-0.065	-1.097	-3.191	99.16	1.48	24.99	72.68
-514.590	-6.490	-0.30	-6.319	-0.095	-1.708	-4.515	97.36	1.46	26.32	69.57
-572.760	-8.740	-0.40	-8.249	-0.018	-2.777	-5.455	94.38	0.20	31.77	62.41
-662.060	-10.800	-0.49	-10.649	-0.183	-3.539	-6.927	98.60	1.69	32.77	64.14
-732.090	-13.000	-0.59	-12.784	-0.163	-4.494	-8.127	98.34	1.25	34.57	62.51
-780.050	-15.500	-0.70	-15.363	-0.155	-6.206	-9.002	99.12	1.00	40.04	58.07
-818.230	-17.550	-0.80	-17.371	-0.198	-7.273	-9.900	98.98	1.13	41.44	56.41
-843.610	-19.700	-0.90	-19.910	-0.340	-9.023	-10.548	101.07	1.73	45.80	53.54
-854.020	-21.740	-0.99	-22.592	-0.400	-11.027	-11.166	103.92	1.84	50.72	51.36
-809.300	-26.560	-1.21	-30.666	-0.425	-18.342	-11.899	115.46	1.60	69.06	44.80
-378.220	-31.120	-1.41	-44.205	-0.465	-36.665	-7.075	142.05	1.49	117.82	22.73
-378.450	-35.190	-1.60	-50.463	-0.150	-41.995	-8.318	143.40	0.43	119.34	23.64

Appendix A: Experimental Data

Table A.93 Vertical strain profile at $y = 0$ mm; $z = 50$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-1	0	-2	-3	-1	-2	-1
0.05		-342	-183	-63	-44	1	50	89
0.10		-448	-209	17	147	491	797	563
0.22		-574	-102	73	290	922	1018	886
0.30		-654	-27	46	483	1399	1238	1208
0.40		-785	60	83	704	2009	1408	1546
0.50		-877	174	80	869	2517	1580	1931
0.59		-939	248	89	1049	2849	1657	2256
0.70		-1011	429	82	1234	3039	1667	2558
0.80		-874	546	80	1383	404	1602	2822
0.89		-813	681	83	1511	273	1416	2746
1.00		-910	1012	55	1522	256	1402	2561
1.17		-992	1291	17	1560	264	1317	991
1.20		-1070	1317	-8	1525	241	1239	936
1.26		-876	1404	-63	455	-43	850	561
1.39		-841	1526	-78	122	-145	764	477
1.63		-1083	1315	-80	-406	-247	441	338

Table A.94 Vertical strain profile at $y = 0$ mm; $z = 950$ mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	0	-1	0	-2	-3
0.05		0	-313	-618	-337	-289	-50	3
0.10		0	-339	-663	-378	-292	-21	38
0.22		0	-360	-791	-572	-229	237	647
0.30		0	-398	-828	-478	-146	497	1030
0.40		0	-418	-881	34	910	718	1445
0.50		0	-433	-978	712	3023	882	1860
0.59		0	-424	-1078	838	11	950	2201
0.70		0	-429	-1213	237	-82	517	2512
0.80		0	-364	-1394	-32	-87	554	2538
0.89		0	-352	-1506	158	-117	551	2365
1.00		0	-340	-1658	-911	-221	540	2392
1.17		0	-334	-1597	-542	-302	543	2548
1.20		0	-334	-1423	-541	-309	520	2533
1.26		0	-347	-1052	-742	-549	-60	2241
1.39		0	-324	-1272	-733	-500	-265	2341
1.63		0	-302	-950	0	-455	-643	2037

Table A.95 Vertical strain profile at y = 0 mm; z = 1950 mm.

DRIFT (%)	x = (mm)	-462	-400	-200	0	200	400	462
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		0	-1	-1	-2	-2	-2	-3
0.05		0	-65	-106	-124	-104	-87	-28
0.10		0	-61	-102	-117	-98	-80	-28
0.22		0	-62	-121	-153	-139	-80	-33
0.30		0	-55	-128	-172	-171	-89	-30
0.40		0	-51	-133	-184	-190	-107	-27
0.50		0	-65	-68	-175	-217	-148	-27
0.59		0	-66	-47	-174	-243	-150	6
0.70		0	-48	-10	-173	-315	48	96
0.80		0	-49	-68	-173	-210	275	133
0.89		0	-50	-51	-187	-176	541	158
1.00		0	-49	-2	-176	-156	869	185
1.17		0	-50	92	-177	-198	1187	242
1.20		0	-49	93	-177	-197	1246	257
1.26		0	-20	312	-178	251	1340	189
1.39		0	-14	239	-217	1628	1418	171
1.63		0	-4	437	0	2261	1408	117

Table A.96 Vertical strain profile at x = -462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212	52	132	212
		(μ)	(μ)	(μ)	(μ)
0.00		1	-1	-1	-1
0.05		-464	-342	-296	-354
0.10		-579	-448	-379	-449
0.22		-724	-574	-467	-538
0.30		-808	-654	-525	-610
0.40		-878	-785	-610	-708
0.50		-961	-877	-686	-790
0.59		-1017	-939	-749	-905
0.70		-1068	-1011	-822	-587
0.80		-1123	-874	-888	-360
0.89		-1375	-813	-974	-331
1.00		-1668	-910	-1262	-383
1.17		-1892	-992	-1663	-415
1.20		-1866	-1070	-1945	-467
1.26		-1382	-876	-1747	-296
1.39		-1357	-841	-1702	-258
1.63		-1336	-1083	-1856	-303

Table A.97 Vertical strain profile at x = -462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		0	0	-1	0
0.05		-260	0	-234	-253
0.10		-291	0	-255	-280
0.22		-301	0	-254	-279
0.30		-280	0	-213	-228
0.40		-295	0	-224	-231
0.50		-307	0	-231	-239
0.59		-317	0	-239	-265
0.70		-322	0	-209	-258
0.80		-357	0	-214	-307
0.89		-342	0	-214	-319
1.00		-322	0	-201	-338
1.17		-283	0	-198	-360
1.20		-287	0	-182	-341
1.26		-151	0	-50	-197
1.39		-200	0	-123	-273
1.63		-223	0	-117	-259

Table A.98 Vertical strain profile at x = -462 mm; z = 1950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-1	0	-2	-1
0.05		-96	0	-65	-81
0.10		-95	0	-62	-79
0.22		-99	0	-54	-69
0.30		-100	0	-49	-62
0.40		-104	0	-49	-62
0.50		-107	0	-45	-54
0.59		-98	0	-41	-41
0.70		-77	0	-17	-6
0.80		-68	0	-4	12
0.89		-50	0	10	26
1.00		-51	0	15	28
1.17		-56	0	19	29
1.20		-59	0	21	32
1.26		-46	0	35	57
1.39		-29	0	47	67
1.63		-19	0	67	98

Table A.99 Vertical strain profile at x = 462 mm; z = 50 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-1	-1	-2	-1
0.05		36	89	63	91
0.10		556	563	334	367
0.22		819	886	529	504
0.30		1132	1208	740	718
0.40		1501	1546	960	955
0.50		1840	1931	1167	1160
0.59		2092	2256	1307	1276
0.70		2292	2558	1403	1390
0.80		2434	2822	1505	1444
0.89		2494	2746	1544	1564
1.00		2528	2561	1589	1681
1.17		2657	991	973	1960
1.20		2695	936	918	1593
1.26		1547	561	492	518
1.39		1339	477	413	417
1.63		772	338	222	130

Table A.100 Vertical strain profile at x = 462 mm; z = 950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-2	-3	-3	-2
0.05		-44	3	10	34
0.10		11	38	43	58
0.22		472	647	849	930
0.30		861	1030	1199	1271
0.40		1258	1445	1672	1659
0.50		1618	1860	2370	2041
0.59		1907	2201	2750	2285
0.70		2176	2512	3254	2534
0.80		2434	2538	3496	2763
0.89		2605	2365	3700	2944
1.00		2768	2392	3913	3113
1.17		3047	2548	4266	3395
1.20		3080	2533	4294	3426
1.26		2691	2241	4019	3033
1.39		2813	2341	4163	3082
1.63		2906	2037	4076	3076

Table A.101 Vertical strain profile at x = 462 mm; z = 1950 mm.

DRIFT (%)	y = (mm)	-212 (μ)	52 (μ)	132 (μ)	212 (μ)
0.00		-3	-3	-2	-1
0.05		-74	-28	-38	-31
0.10		-69	-28	-41	-36
0.22		-65	-33	-45	-35
0.30		-46	-30	-36	-28
0.40		-22	-27	-8	42
0.50		55	-27	42	322
0.59		183	6	104	497
0.70		440	96	210	698
0.80		581	133	270	807
0.89		625	158	319	883
1.00		698	185	383	967
1.17		802	242	497	1069
1.20		801	257	504	1072
1.26		567	189	383	810
1.39		510	171	355	754
1.63		380	117	267	584

Table A.102 Horizontal strain profile at $x = -475$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-2	-1	-2	0	-2	-1	-2	-1	-2	-2	-1
0.05		57	176	140	107	83	38	19	7	11	-6	-1
0.10		83	227	171	126	95	47	26	12	15	-5	0
0.22		123	319	226	144	99	49	26	12	14	-3	2
0.30		154	373	255	191	131	104	40	12	11	-6	6
0.40		194	447	300	226	162	133	51	50	13	3	6
0.50		231	529	343	257	191	169	62	86	8	34	9
0.59		279	583	389	292	224	217	79	115	46	49	9
0.70		214	647	433	324	238	278	98	147	90	76	6
0.80		257	729	487	335	229	319	117	175	118	95	7
0.89		337	870	566	344	244	354	131	192	142	116	7
1.00		514	1049	869	478	245	366	145	224	170	137	8
1.17		709	1255	1183	590	246	371	155	258	208	151	13
1.20		743	1317	1309	625	249	369	154	255	208	151	14
1.26		550	1056	864	462	280	517	235	483	327	147	30
1.39		502	1040	823	459	285	515	278	630	372	150	29
1.63		266	0	913	419	242	462	308	1033	545	167	31

Table A.103 Horizontal strain profile at x = -350 mm; y = 0 mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-1	0	-1	0	0	-1	-1	0	0	-1	0
0.05		-76	-23	35	0	23	36	-2	0	11	35	12
0.10		-71	-40	28	0	22	41	-6	0	10	40	12
0.22		271	59	118	0	18	40	-3	0	13	44	14
0.30		332	244	497	0	103	436	449	0	216	53	12
0.40		433	402	688	0	448	510	556	0	413	108	14
0.50		498	716	882	0	785	681	703	0	893	1180	28
0.59		571	964	1101	0	1099	676	1134	0	1253	1567	36
0.70		642	1587	1580	0	1302	1289	2000	0	2235	1477	79
0.80		715	1903	2028	0	1904	2542	2450	0	1929	1391	94
0.89		757	2470	2607	0	1959	3010	2513	0	1684	1544	107
1.00		907	2907	3075	0	2111	3252	2578	0	1617	1850	135
1.17		1249	2088	3283	0	2491	3183	2400	0	1433	2147	182
1.20		1293	2098	3244	0	2549	3146	2360	0	1430	2151	184
1.26		1283	801	3191	0	2822	3209	1646	0	1298	2302	257
1.39		1266	564	3180	0	2952	3309	2530	0	1245	2288	259
1.63		1006	385	3061	0	2712	2676	2472	0	1211	2369	268

Table A.104 Horizontal strain profile at $x = -50$ mm; $y = 0$ mm.

DRIFT (%)	$z =$ (mm)	0	400	800	1200	1600	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-3	-1	0	-1	-1	0
0.05		-38	-12	28	28	41	0
0.10		-110	-47	31	30	44	0
0.22		-20	1188	1106	38	54	0
0.30		-31	1480	1913	75	52	0
0.40		-60	1780	2224	139	60	0
0.50		-86	2013	2732	652	54	0
0.59		-117	2137	2879	1071	62	0
0.70		-102	2454	2983	4847	366	0
0.80		-99	2563	3005	1369	2508	0
0.89		-108	2631	3185	1477	2676	0
1.00		-104	2616	0	1657	2867	0
1.17		-14	0	0	1213	3651	0
1.20		-4	0	0	1213	3689	0
1.26		100	0	0	1350	3838	0
1.39		135	0	0	1353	2330	0
1.63		211	0	0	1292	1738	0

Table A.105 Horizontal strain profile at $x = 350$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
		(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)	(μ)
0.00		-3	-3	-2	-1	0	0	0	-1	-2	-2	0
0.05		1	2	-5	-12	5	11	17	13	14	16	12
0.10		-129	19	284	8	12	20	22	16	17	18	1
0.22		-81	18	576	933	994	1584	1400	1394	734	79	-10
0.30		99	364	693	1120	1256	1778	1647	1851	1673	111	-16
0.40		235	395	828	1350	1589	2108	2132	2281	2143	303	-7
0.50		270	552	1239	1619	1877	2442	2543	2641	2667	572	-3
0.59		317	604	1317	1690	2051	2594	2688	2850	3264	1101	14
0.70		392	724	1628	1860	2226	2699	2708	2930	3135	2540	89
0.80		444	835	1692	1897	2266	2708	2774	2987	3128	2854	134
0.89		485	881	1707	1862	2536	2724	2859	3124	17331	3228	176
1.00		517	958	1654	1890	2923	2726	2911	3330	11842	3563	251
1.17		520	3437	5168	1791	3097	2870	3050	3715	11445	4019	317
1.20		517	3429	5147	1776	3177	2917	3103	4046	11477	4110	322
1.26		584	1368	7283	913	2330	3405	1759	941	11176	3539	448
1.39		602	1376	7306	913	2629	3425	2169	1077	11236	3565	463
1.63		848	1559	7313	1040	5127	1029	1739	859	11178	3341	504

Table A.106 Horizontal strain profile at $x = 475$ mm; $y = 0$ mm.

DRIFT (%)	z = (mm)	0 (μ)	200 (μ)	400 (μ)	600 (μ)	800 (μ)	1000 (μ)	1200 (μ)	1400 (μ)	1600 (μ)	1800 (μ)	2000 (μ)
0.00		0	-1	0	0	-1	-1	1	-2	-1	-1	186
0.05		-17	-11	0	9	5	15	2	4	14	18	91962
0.10		-69	-75	0	43	22	26	7	10	18	27	46870
0.22		-15	-44	0	107	99	197	83	10	0	17	0
0.30		12	-1	0	135	144	197	157	135	1	50	23461
0.40		74	19	0	162	193	235	173	294	114	102	45126
0.50		109	73	0	194	236	320	196	474	289	126	0
0.59		143	123	0	234	267	403	210	591	346	207	-41542
0.70		169	176	0	285	282	467	208	662	394	309	0
0.80		201	231	0	317	290	515	214	677	446	406	-32308
0.89		239	283	0	346	303	554	224	662	467	483	-122754
1.00		284	360	0	368	317	590	235	645	500	592	-14618
1.17		315	421	0	395	340	683	270	648	568	630	0
1.20		317	425	0	398	342	690	278	656	586	615	0
1.26		321	410	0	404	315	665	337	570	614	497	0
1.39		326	409	0	426	312	754	488	589	644	488	0
1.63		332	384	0	495	836	1141	436	502	683	464	0

APPENDIX B

PROPOSED MODELS CALCULATION

In this part, specimens data from past experiments on RC walls and this experimental study that have not been shown in previous chapters are presented here in table form. Furthermore, the calculations of shear strengths of RC walls using the proposed models are also presented here in table form.

Appendix B: Proposed Models Calculation

Table B.1 Data of RC walls failing in shear mode collected from past experiments and the author's experimental study.

No.	Specimen ID	A_g (mm ²)	H_w (mm)	L_w (mm)	t_w (mm)	b_f (mm)	t_f (mm)	ρ_b	f_{yb} (MPa)	ρ_v	f_{yv} (MPa)	ρ_h	f_{yh} (MPa)
(Barda et al. 1977)													
1	B1-1	296774	876	1905	102	610	102	0.0180	525	0.0050	543	0.0050	496
2	B2-1	296774	876	1905	102	610	102	0.0640	487	0.0050	552	0.0050	499
3	B3-2	296774	876	1905	102	610	102	0.0410	414	0.0050	545	0.0050	513
4	B6-4	296774	876	1905	102	610	102	0.0410	529	0.0025	496	0.0050	496
5	B7-5	296774	400	1905	102	610	102	0.0410	539	0.0050	531	0.0050	501
6	B8-5	296774	1829	1905	102	610	102	0.0410	489	0.0050	527	0.0050	496
(Cardenas et al. 1980)													
7	SW-7	145161	1905	1905	76	76	191	0.0767	448	0.0077	448	0.0027	414
8	SW-8	145161	1905	1905	76	76	191	0.0300	448	0.0300	448	0.0027	465
(Corley et al. 1981)													
9	B2	317419	4572	1905	102	305	305	0.0367	410	0.0029	532	0.0063	532
10	B5	317419	4572	1905	102	305	305	0.0367	444	0.0029	502	0.0063	502
11	B6	317419	4572	1905	102	305	305	0.0367	441	0.0029	512	0.0063	512
12	B7	317419	4572	1905	102	305	305	0.0367	458	0.0029	490	0.0063	490
13	B8	317419	4572	1905	102	305	305	0.0367	447	0.0029	454	0.0138	482
14	B9	317419	4572	1905	102	305	305	0.0367	430	0.0029	461	0.0063	461
15	B10	317419	4572	1905	102	305	305	0.0197	443	0.0029	464	0.0063	464
16	F1	358709	4572	1905	102	914	102	0.0389	445	0.0030	525	0.0071	525
17	F2	358709	4572	1905	102	914	102	0.0435	430	0.0031	464	0.0063	464
(Maeda 1986)													
18	MAE03	210400	1200	2180	80	180	180	0.0781	389	0.0119	321	0.0119	321
19	MAE07	210400	1200	2180	80	180	180	0.0781	389	0.0200	321	0.0200	321
(Okamoto 1990)													
20	W48M6	369600	1280	1720	120	800	120	0.0089	560	0.0079	560	0.0079	560
21	W48M4	369600	1280	1720	120	800	120	0.0119	347	0.0119	347	0.0119	347

Appendix B: Proposed Models Calculation

Table B.1 Data of RC walls failing in shear mode collected from past experiments and the author's experimental study (continued).

No.	Specimen ID	A_g (mm ²)	H_w (mm)	L_w (mm)	t_w (mm)	b_f (mm)	t_f (mm)	ρ_b	f_{yb} (MPa)	ρ_v	f_{yv} (MPa)	ρ_h	f_{yh} (MPa)
22	W72M8	369600	1280	1720	120	800	120	0.0089	792	0.0091	792	0.0091	792
23	W72M6	369600	1280	1720	120	800	120	0.0119	560	0.0119	560	0.0119	560
24	W72M8	369600	1280	1720	120	800	120	0.0089	792	0.0091	792	0.0091	792
25	W96M8	369600	1280	1720	120	800	120	0.0119	792	0.0119	792	0.0119	792
(Mo and Chan 1996)													
26	HN4-1	76200	500	860	70	170	80	0.0210	302	0.0073	302	0.0081	302
27	HN4-2	76200	500	860	70	170	80	0.0210	302	0.0073	302	0.0081	302
28	HN4-3	76200	500	860	70	170	80	0.0210	302	0.0073	302	0.0081	302
29	HN6-1	76200	500	860	70	170	80	0.0210	443	0.0073	443	0.0081	443
30	HN6-2	76200	500	860	70	170	80	0.0210	443	0.0073	443	0.0081	443
31	HN6-3	76200	500	860	70	170	80	0.0210	443	0.0073	443	0.0081	443
32	HM4-1	76200	500	860	70	170	80	0.0210	302	0.0073	302	0.0081	302
33	HM4-2	76200	500	860	70	170	80	0.0210	302	0.0073	302	0.0081	302
34	HM4-3	76200	500	860	70	170	80	0.0210	302	0.0073	302	0.0081	302
35	LN4-1	76200	500	860	70	170	80	0.0210	302	0.0058	302	0.0081	302
36	LN4-2	76200	500	860	70	170	80	0.0210	302	0.0058	302	0.0081	302
37	LN4-3	76200	500	860	70	170	80	0.0210	302	0.0058	302	0.0081	302
38	LN6-1	76200	500	860	70	170	80	0.0210	443	0.0058	443	0.0081	443
39	LN6-2	76200	500	860	70	170	80	0.0210	443	0.0058	443	0.0081	443
40	LN6-3	76200	500	860	70	170	80	0.0210	443	0.0058	443	0.0081	443
41	LM6-1	76200	500	860	70	170	80	0.0210	443	0.0058	443	0.0081	443
42	LM6-2	76200	500	860	70	170	80	0.0210	443	0.0058	443	0.0081	443
43	LM6-3	76200	500	860	70	170	80	0.0210	443	0.0058	443	0.0081	443
44	LM4-2	76200	500	860	70	170	80	0.0210	302	0.0058	302	0.0081	302
45	LM4-3	76200	500	860	70	170	80	0.0210	302	0.0058	302	0.0081	302
(Gupta and Rangan 1998)													

Appendix B: Proposed Models Calculation

Table B.1 Data of RC walls failing in shear mode collected from past experiments and the author's experimental study (continued).

No.	Specimen ID	A_g (mm ²)	H_w (mm)	L_w (mm)	t_w (mm)	b_f (mm)	t_f (mm)	ρ_b	f_{yb} (MPa)	ρ_v	f_{yv} (MPa)	ρ_h	f_{yh} (MPa)
46	S-1	135000	1000	1000	75	375	100	0.0210	535	0.0100	545	0.0050	578
47	S-2	135000	1000	1000	75	375	100	0.0304	535	0.0100	545	0.0050	578
48	S-3	135000	1000	1000	75	375	100	0.0387	535	0.0100	545	0.0050	578
49	S-4	135000	1000	1000	75	375	100	0.0315	535	0.0150	533	0.0050	578
50	S-5	135000	1000	1000	75	375	100	0.0399	535	0.0150	533	0.0050	578
51	S-6	135000	1000	1000	75	375	100	0.0446	535	0.0150	533	0.0050	578
52	S-7	135000	1000	1000	75	375	100	0.0304	535	0.0100	545	0.0100	545
(Kabeyasawa and Hiraishi 1998)													
53	W-08	184000	2000	1700	80	200	200	0.0214	761	0.0053	1079	0.0053	1079
54	W-12	184000	2000	1700	80	200	200	0.0214	761	0.0053	1079	0.0053	1079
55	No. 1	184000	2000	1700	80	200	200	0.0508	1009	0.0020	792	0.0020	792
56	No. 2	184000	2000	1700	80	200	200	0.0508	1009	0.0035	792	0.0035	792
57	No. 3	184000	2000	1700	80	200	200	0.0508	1009	0.0053	792	0.0053	792
58	No. 4	184000	2000	1700	80	200	200	0.0508	1009	0.0053	792	0.0053	792
59	No. 5	184000	3000	1700	80	200	200	0.0508	1009	0.0053	792	0.0053	792
60	No. 6	184000	2000	1700	80	200	200	0.0508	1009	0.0066	1420	0.0066	1420
61	No. 7	184000	2000	1700	80	200	200	0.0508	1009	0.0100	792	0.0100	792
62	No. 8	184000	2000	1700	80	200	200	0.0508	1009	0.0145	792	0.0145	792
(Farvashany et al. 2008)													
63	HSCW1	120000	1100	880	75	375	90	0.0400	670	0.0126	535	0.0047	535
64	HSCW2	120000	1100	880	75	375	90	0.0400	670	0.0126	535	0.0047	535
65	HSCW3	120000	1100	880	75	375	90	0.0400	670	0.0075	535	0.0047	535
66	HSCW4	120000	1100	880	75	375	90	0.0400	670	0.0075	535	0.0047	535
67	HSCW5	120000	1100	880	75	375	90	0.0400	670	0.0126	535	0.0075	535
68	HSCW6	120000	1100	880	75	375	90	0.0400	670	0.0126	535	0.0075	535
69	HSCW7	120000	1100	880	75	375	90	0.0400	670	0.0075	535	0.0075	535

Appendix B: Proposed Models Calculation

Table B.1 Data of RC walls failing in shear mode collected from past experiments and the author's experimental study (continued).

No.	Specimen ID	A_g (mm ²)	H_w (mm)	L_w (mm)	t_w (mm)	b_f (mm)	t_f (mm)	ρ_b	f_{yb} (MPa)	ρ_v	f_{yv} (MPa)	ρ_h	f_{yh} (MPa)
(Burgueno et al. 2014)													
70	M05C	167640	2286	1016	76	254	254	0.0556	491	0.0147	445	0.0183	445
71	M05M	167640	2286	1016	76	254	254	0.0556	491	0.0147	445	0.0183	445
72	M10C	167640	2286	1016	76	254	254	0.0556	457	0.0147	476	0.0183	476
73	M10M	167640	2286	1016	76	254	254	0.0556	457	0.0147	476	0.0183	476
74	M15C	167640	2286	1016	76	254	254	0.0528	439	0.0147	481	0.0183	481
75	M15M	167640	2286	1016	76	254	254	0.0556	514	0.0147	478	0.0183	478
76	M20C	167640	2286	1016	76	254	254	0.0556	449	0.0147	438	0.0244	438
77	M20M	167640	2286	1016	76	254	254	0.0556	449	0.0147	438	0.0244	438
Author													
78	J1	196000	1000	1000	100	500	120	0.0388	630	0.0028	610	0.0028	610
79	J2	196000	1000	1000	100	500	120	0.0388	630	0.0075	578	0.0028	610
80	J3	196000	1000	1000	100	500	120	0.0388	630	0.0028	610	0.0075	578
81	J4	111200	1000	1000	100	120	280	0.0693	630	0.0028	610	0.0028	610
82	J5	196000	2000	1000	100	500	120	0.0388	630	0.0028	610	0.0028	610
83	J6	196000	2000	1000	100	500	120	0.0388	630	0.0075	578	0.0028	610
84	J7	196000	2000	1000	100	500	120	0.0388	630	0.0028	610	0.0075	578

Appendix B: Proposed Models Calculation

Table B.2 Calculation of shear strengths of RC walls using the complete truss model.

No.	Specimen ID	ε_d	ε_r	ζ	σ_r (MPa)	σ_d (MPa)	ε_v	ε_h	f_v (MPa)	f_h (MPa)	θ (deg)	D_u (kN)	V_n (kN)
(Barda et al. 1977)													
1	B1-1	-0.0003	0.0732	0.16	0.58	-4.74	0.0351	0.0378	434.65	396.59	46.03	90.04	576.60
2	B2-1	-0.0005	0.0239	0.28	0.33	-4.53	0.0111	0.0122	441.26	399.34	46.24	131.56	576.08
3	B3-2	-0.0004	0.0625	0.18	0.54	-4.77	0.0303	0.0318	435.75	410.38	45.68	128.05	614.53
4	B6-4	-0.0003	0.0764	0.16	0.42	-3.40	0.0480	0.0281	397.14	397.14	37.48	118.15	456.85
5	B7-5	-0.0004	0.0596	0.18	0.51	-4.64	0.0289	0.0303	424.72	401.00	45.66	50.65	523.00
6	B8-5	-0.0004	0.0510	0.19	0.47	-4.56	0.0246	0.0259	421.96	396.59	45.72	147.52	608.23
(Cardenas et al. 1980)													
7	SW-7	-0.0002	0.1760	0.10	0.86	-4.50	0.0574	0.1183	358.54	330.96	55.12	8.34	337.06
8	SW-8	-0.0007	0.0200	0.30	0.85	-12.61	0.0022	0.0171	358.54	372.33	68.21	7.09	613.23
(Corley et al. 1981)													
9	B2	-0.0002	0.1679	0.10	1.07	-5.14	0.1014	0.0664	425.84	425.84	39.00	97.36	591.73
10	B5	-0.0002	0.1660	0.10	0.91	-4.75	0.1007	0.0651	401.56	401.56	38.82	92.03	541.52
11	B6	-0.0007	0.0156	0.33	0.44	-7.28	0.0057	0.0093	409.29	409.29	51.33	72.10	684.41
12	B7	-0.0004	0.0550	0.17	0.99	-8.50	0.0198	0.0348	391.64	391.64	52.88	94.70	836.92
13	B8	-0.0006	0.0260	0.26	0.84	-11.12	0.0131	0.0123	362.95	385.57	44.14	89.73	1061.74
14	B9	-0.0004	0.0529	0.19	0.88	-8.19	0.0184	0.0340	369.02	369.02	53.51	91.23	796.21
15	B10	-0.0004	0.0541	0.18	0.91	-8.24	0.0189	0.0347	371.23	371.23	53.41	72.58	784.63
16	F1	-0.0003	0.1105	0.13	0.77	-5.15	0.0699	0.0402	420.32	420.32	37.23	317.94	840.49
17	F2	-0.0004	0.0595	0.17	0.91	-7.86	0.0218	0.0373	371.23	371.23	52.50	354.10	1130.33
(Maeda 1986)													
18	MAE03	-0.0004	0.0547	0.16	1.17	-9.26	0.0220	0.0323	256.80	256.80	50.41	44.48	863.45
19	MAE07	-0.0006	0.0247	0.23	1.16	-13.39	0.0104	0.0138	256.80	256.80	48.87	44.43	1198.18
(Okamoto 1990)													
20	W48M6	-0.0004	0.0579	0.13	1.65	-10.71	0.0241	0.0334	448.00	448.00	49.57	167.03	1337.76
21	W48M4	-0.0003	0.0640	0.12	1.65	-10.21	0.0265	0.0371	277.60	277.60	49.76	191.55	1313.57

Table B.2 Calculation of shear strengths of RC walls using the complete truss model (continued).

No.	Specimen ID	ε_d	ε_r	ζ	σ_r (MPa)	σ_d (MPa)	ε_v	ε_h	f_v (MPa)	f_h (MPa)	θ (deg)	D_u (kN)	V_n (kN)
22	W72M8	-0.0004	0.0401	0.15	1.65	-12.75	0.0171	0.0226	500.00	500.00	48.92	167.03	1536.02
23	W72M6	-0.0005	0.0316	0.17	1.65	-14.26	0.0136	0.0175	448.00	448.00	48.54	191.55	1706.39
24	W72M8	-0.0004	0.0471	0.13	2.04	-13.14	0.0203	0.0264	500.00	500.00	48.71	179.30	1623.84
25	W96M8	-0.0005	0.0314	0.16	2.04	-15.88	0.0137	0.0172	500.00	500.00	48.14	205.62	1915.30
(Mo and Chan 1996)													
26	HN4-1	-0.0003	0.1074	0.14	0.64	-4.37	0.0558	0.0513	241.84	241.84	43.79	15.00	151.81
27	HN4-2	-0.0003	0.1074	0.14	0.64	-4.37	0.0558	0.0513	241.84	241.84	43.79	15.00	151.81
28	HN4-3	-0.0003	0.1068	0.14	0.64	-4.37	0.0555	0.0510	241.84	241.84	43.79	14.99	151.69
29	HN6-1	-0.0004	0.0456	0.21	0.59	-6.06	0.0236	0.0215	354.56	354.56	43.67	14.57	195.79
30	HN6-2	-0.0004	0.0456	0.21	0.59	-6.06	0.0236	0.0215	354.56	354.56	43.67	14.57	195.79
31	HN6-3	-0.0004	0.0501	0.20	0.62	-6.09	0.0260	0.0237	354.56	354.56	43.68	14.81	197.68
32	HM4-1	-0.0003	0.1395	0.12	0.75	-4.48	0.0724	0.0668	241.84	241.84	43.84	15.79	158.39
33	HM4-2	-0.0003	0.1395	0.12	0.75	-4.48	0.0724	0.0668	241.84	241.84	43.84	15.79	158.39
34	HM4-3	-0.0003	0.1549	0.11	0.80	-4.53	0.0804	0.0742	241.84	241.84	43.86	16.11	161.34
35	LN4-1	-0.0004	0.0445	0.21	0.36	-3.74	0.0251	0.0190	241.84	241.84	41.05	12.36	123.12
36	LN4-2	-0.0004	0.0445	0.21	0.36	-3.74	0.0251	0.0190	241.84	241.84	41.05	12.36	123.12
37	LN4-3	-0.0003	0.1108	0.13	0.59	-3.97	0.0621	0.0484	241.84	241.84	41.46	14.60	138.26
38	LN6-1	-0.0004	0.0592	0.18	0.61	-5.56	0.0334	0.0254	354.56	354.56	41.16	14.77	181.91
39	LN6-2	-0.0004	0.0574	0.18	0.60	-5.55	0.0324	0.0246	354.56	354.56	41.15	14.69	181.28
40	LN6-3	-0.0004	0.0574	0.18	0.60	-5.55	0.0324	0.0246	354.56	354.56	41.15	14.69	181.28
41	LM6-1	-0.0003	0.0926	0.15	0.79	-5.74	0.0520	0.0402	354.56	354.56	41.37	16.03	192.65
42	LM6-2	-0.0003	0.0831	0.15	0.74	-5.69	0.0468	0.0360	354.56	354.56	41.31	15.71	189.80
43	LM6-3	-0.0004	0.0733	0.16	0.69	-5.64	0.0413	0.0317	354.56	354.56	41.26	15.35	186.68
44	LM4-2	-0.0002	0.2492	0.07	1.32	-4.70	0.1362	0.1128	241.84	241.84	42.32	19.06	182.58
45	LM4-3	-0.0002	0.2492	0.07	1.32	-4.70	0.1362	0.1128	241.84	241.84	42.32	19.06	182.58
(Gupta and Rangan 1998)													

Table B.2 Calculation of shear strengths of RC walls using the complete truss model (continued).

No.	Specimen ID	ε_d	ε_r	ζ	σ_r (MPa)	σ_d (MPa)	ε_v	ε_h	f_v (MPa)	f_h (MPa)	θ (deg)	D_u (kN)	V_n (kN)
46	S-1	-0.0003	0.0953	0.10	1.59	-8.26	0.0376	0.0574	436.00	462.40	51.00	76.55	401.52
47	S-2	-0.0005	0.0326	0.19	1.30	-12.49	0.0082	0.0239	436.00	462.40	59.21	84.63	494.06
48	S-3	-0.0006	0.0172	0.25	1.38	-17.16	0.0029	0.0136	436.00	462.40	63.50	96.03	595.85
49	S-4	-0.0004	0.0581	0.14	1.50	-10.21	0.0187	0.0391	426.56	462.40	55.20	90.25	460.92
50	S-5	-0.0005	0.0260	0.20	1.46	-14.69	0.0057	0.0198	426.56	462.40	61.09	99.13	560.50
51	S-6	-0.0007	0.0141	0.27	1.41	-18.88	0.0020	0.0114	403.26	462.40	64.64	102.77	632.86
52	S-7	-0.0005	0.0254	0.21	1.42	-14.66	0.0088	0.0160	436.00	436.00	53.16	87.19	608.25
(Kabeyasawa and Hiraishi 1998)													
53	W-08	-0.0005	0.0276	0.16	2.07	-16.99	0.0065	0.0206	500.00	500.00	60.11	42.41	1030.28
54	W-12	-0.0005	0.0246	0.15	2.75	-20.65	0.0053	0.0188	500.00	500.00	61.24	46.66	1231.14
55	No. 1	-0.0005	0.0367	0.18	1.30	-11.82	0.0060	0.0302	500.00	500.00	65.24	50.05	649.04
56	No. 2	-0.0005	0.0306	0.19	1.42	-13.42	0.0061	0.0240	500.00	500.00	62.53	51.47	779.84
57	No. 3	-0.0006	0.0233	0.21	1.44	-15.29	0.0053	0.0175	500.00	500.00	60.31	51.71	915.32
58	No. 4	-0.0006	0.0161	0.21	2.07	-21.62	0.0027	0.0128	500.00	500.00	63.45	58.39	1195.28
59	No. 5	-0.0005	0.0247	0.20	1.53	-15.39	0.0057	0.0185	500.00	500.00	60.12	52.86	930.11
60	No. 6	-0.0006	0.0202	0.22	1.48	-16.56	0.0049	0.0147	500.00	500.00	59.09	52.25	1006.49
61	No. 7	-0.0007	0.0126	0.28	1.43	-19.95	0.0033	0.0086	500.00	500.00	56.74	51.63	1228.24
62	No. 8	-0.0009	0.0085	0.32	1.52	-24.14	0.0023	0.0053	468.97	500.00	54.17	52.72	1514.18
(Farvashany et al. 2008)													
63	HSCW1	-0.0004	0.0422	0.13	2.08	-13.98	0.0104	0.0314	428.00	428.00	59.69	106.65	521.35
64	HSCW2	-0.0005	0.0239	0.19	1.86	-17.21	0.0044	0.0189	428.00	428.00	63.22	102.75	557.31
65	HSCW3	-0.0005	0.0302	0.17	1.72	-14.88	0.0064	0.0233	428.00	428.00	61.70	100.10	510.74
66	HSCW4	-0.0008	0.0100	0.27	1.82	-24.70	0.0008	0.0085	156.04	428.00	67.66	102.01	654.46
67	HSCW5	-0.0006	0.0187	0.22	1.68	-18.24	0.0041	0.0140	428.00	428.00	60.30	99.32	607.29
68	HSCW6	-0.0005	0.0312	0.17	1.80	-14.99	0.0090	0.0217	428.00	428.00	56.89	101.63	556.73
69	HSCW7	-0.0005	0.0294	0.16	2.04	-16.39	0.0080	0.0209	428.00	428.00	57.75	105.96	598.89

Table B.2 Calculation of shear strengths of RC walls using the complete truss model (continued).

No.	Specimen ID	ε_d	ε_r	ζ	σ_r (MPa)	σ_d (MPa)	ε_v	ε_h	f_v (MPa)	f_h (MPa)	θ (deg)	D_u (kN)	V_n (kN)
(Burgueno et al. 2014)													
70	M05C	-0.0008	0.0124	0.35	0.92	-16.12	0.0049	0.0066	356.00	356.00	48.66	74.03	563.47
71	M05M	-0.0009	0.0095	0.41	0.78	-15.98	0.0036	0.0050	356.00	356.00	48.72	70.01	551.15
72	M10C	-0.0007	0.0136	0.30	1.13	-17.15	0.0056	0.0073	380.80	380.80	48.27	79.24	604.99
73	M10M	-0.0006	0.0200	0.21	1.68	-17.70	0.0086	0.0108	380.80	380.80	48.08	90.49	648.41
74	M15C	-0.0005	0.0234	0.18	2.04	-18.19	0.0102	0.0127	384.80	384.80	47.93	94.94	677.72
75	M15M	-0.0005	0.0254	0.16	2.22	-18.29	0.0111	0.0138	382.40	382.40	47.91	99.30	690.22
76	M20C	-0.0005	0.0257	0.15	2.62	-19.77	0.0125	0.0126	350.40	350.40	45.07	104.93	753.39
77	M20M	-0.0005	0.0231	0.17	2.30	-19.45	0.0112	0.0113	350.40	350.40	45.07	100.48	730.40
Author													
78	J1	-0.0003	0.0850	0.10	2.07	-9.96	0.0240	0.0607	488.00	488.00	57.71	186.66	664.70
79	J2	-0.0004	0.0579	0.12	1.94	-11.61	0.0138	0.0437	462.40	488.00	60.41	182.66	694.50
80	J3	-0.0004	0.0563	0.11	2.21	-12.58	0.0214	0.0345	488.00	462.40	51.71	191.02	824.34
81	J4	-0.0003	0.0889	0.10	1.87	-9.28	0.0256	0.0630	488.00	488.00	57.40	26.57	390.89
82	J5	-0.0003	0.0850	0.10	2.07	-9.96	0.0240	0.0607	488.00	488.00	57.71	186.66	664.70
83	J6	-0.0004	0.0579	0.12	1.94	-11.61	0.0138	0.0437	462.40	488.00	60.41	182.66	694.50
84	J7	-0.0004	0.0563	0.11	2.21	-12.58	0.0214	0.0345	488.00	462.40	51.71	191.02	824.34

Appendix B: Proposed Models Calculation

Table B.3 Calculation of shear strengths of RC walls using the simplified truss model.

No.	Specimen ID	ζ	σ_r (MPa)	Check	σ_d (MPa)	f_v (MPa)	f_h (MPa)	θ (deg)	D_u (kN)	V_n (kN)
(Barda et al. 1977)										
1	B1-1	0.52	0.58	YIELD	-4.74	434.65	396.59	46.03	90.04	576.60
2	B2-1	0.52	0.33	YIELD	-4.53	441.26	399.34	46.24	131.56	576.08
3	B3-2	0.52	0.54	YIELD	-4.77	435.75	410.38	45.68	128.05	614.53
4	B6-4	0.52	0.42	YIELD	-3.40	397.14	397.14	37.48	118.15	456.85
5	B7-5	0.52	0.51	YIELD	-4.64	424.72	401.00	45.66	50.65	523.00
6	B8-5	0.52	0.47	YIELD	-4.56	421.96	396.59	45.72	147.52	608.23
(Cardenas et al. 1980)										
7	SW-7	0.51	0.86	YIELD	-4.50	358.54	330.96	55.12	8.34	337.06
8	SW-8	0.51	0.85	YIELD	-12.61	358.54	372.33	68.21	7.09	613.23
(Corley et al. 1981)										
9	B2	0.46	1.07	YIELD	-5.14	425.84	425.84	39.00	97.36	591.73
10	B5	0.50	0.91	YIELD	-4.75	401.56	401.56	38.82	92.03	541.52
11	B6	0.52	0.44	YIELD	-7.28	409.29	409.29	51.33	72.10	684.41
12	B7	0.48	0.99	YIELD	-8.50	391.64	391.64	52.88	94.70	836.92
13	B8	0.52	0.84	YIELD	-11.12	362.95	385.57	44.14	89.73	1061.74
14	B9	0.50	0.88	YIELD	-8.19	369.02	369.02	53.51	91.23	796.21
15	B10	0.50	0.91	YIELD	-8.24	371.23	371.23	53.41	72.58	784.63
16	F1	0.52	0.77	YIELD	-5.15	420.32	420.32	37.23	317.94	840.49
17	F2	0.50	0.91	YIELD	-7.86	371.23	371.23	52.50	354.10	1130.33
(Maeda 1986)										
18	MAE03	0.44	1.17	YIELD	-9.26	256.80	256.80	50.41	44.48	863.45
19	MAE07	0.44	1.16	YIELD	-13.39	256.80	256.80	48.87	44.43	1198.18
(Okamoto 1990)										
20	W48M6	0.37	1.65	YIELD	-10.71	448.00	448.00	49.57	167.03	1337.76
21	W48M4	0.37	1.65	YIELD	-10.21	277.60	277.60	49.76	191.55	1313.57

Table B.3 Calculation of shear strengths of RC walls using the simplified truss model (continued).

No.	Specimen ID	ζ	σ_r (MPa)	Check	σ_d (MPa)	f_v (MPa)	f_h (MPa)	θ (deg)	D_u (kN)	V_n (kN)
22	W72M8	0.37	1.65	YIELD	-12.75	500.00	500.00	48.92	167.03	1536.02
23	W72M6	0.37	1.65	YIELD	-14.26	448.00	448.00	48.54	191.55	1706.39
24	W72M8	0.33	2.04	YIELD	-13.14	500.00	500.00	48.71	179.30	1623.84
25	W96M8	0.33	2.04	YIELD	-15.88	500.00	500.00	48.14	205.62	1915.30
(Mo and Chan 1996)										
26	HN4-1	0.52	0.64	YIELD	-4.37	241.84	241.84	43.79	15.00	151.81
27	HN4-2	0.52	0.64	YIELD	-4.37	241.84	241.84	43.79	15.00	151.81
28	HN4-3	0.52	0.64	YIELD	-4.37	241.84	241.84	43.79	14.99	151.69
29	HN6-1	0.52	0.59	YIELD	-6.06	354.56	354.56	43.67	14.57	195.79
30	HN6-2	0.52	0.59	YIELD	-6.06	354.56	354.56	43.67	14.57	195.79
31	HN6-3	0.52	0.62	YIELD	-6.09	354.56	354.56	43.68	14.81	197.68
32	HM4-1	0.52	0.75	YIELD	-4.48	241.84	241.84	43.84	15.79	158.39
33	HM4-2	0.52	0.75	YIELD	-4.48	241.84	241.84	43.84	15.79	158.39
34	HM4-3	0.52	0.80	YIELD	-4.53	241.84	241.84	43.86	16.11	161.34
35	LN4-1	0.52	0.36	YIELD	-3.74	241.84	241.84	41.05	12.36	123.12
36	LN4-2	0.52	0.36	YIELD	-3.74	241.84	241.84	41.05	12.36	123.12
37	LN4-3	0.52	0.59	YIELD	-3.97	241.84	241.84	41.46	14.60	138.26
38	LN6-1	0.52	0.61	YIELD	-5.56	354.56	354.56	41.16	14.77	181.91
39	LN6-2	0.52	0.60	YIELD	-5.55	354.56	354.56	41.15	14.69	181.28
40	LN6-3	0.52	0.60	YIELD	-5.55	354.56	354.56	41.15	14.69	181.28
41	LM6-1	0.52	0.79	YIELD	-5.74	354.56	354.56	41.37	16.03	192.65
42	LM6-2	0.52	0.74	YIELD	-5.69	354.56	354.56	41.31	15.71	189.80
43	LM6-3	0.52	0.69	YIELD	-5.64	354.56	354.56	41.26	15.35	186.68
44	LM4-2	0.41	1.32	YIELD	-4.70	241.84	241.84	42.32	19.06	182.58
45	LM4-3	0.41	1.32	YIELD	-4.70	241.84	241.84	42.32	19.06	182.58
(Gupta and Rangan 1998)										

Table B.3 Calculation of shear strengths of RC walls using the simplified truss model (continued).

No.	Specimen ID	ζ	σ_r (MPa)	Check	σ_d (MPa)	f_v (MPa)	f_h (MPa)	θ (deg)	D_u (kN)	V_n (kN)
46	S-1	0.38	1.59	YIELD	-8.26	436.00	462.40	51.00	76.55	401.52
47	S-2	0.42	1.30	YIELD	-12.49	436.00	462.40	59.21	84.63	494.06
48	S-3	0.40	1.38	YIELD	-17.16	436.00	462.40	63.50	96.03	595.85
49	S-4	0.39	1.50	YIELD	-10.21	426.56	462.40	55.20	90.25	460.92
50	S-5	0.39	1.46	YIELD	-14.69	426.56	462.40	61.09	99.13	560.50
51	S-6	0.40	1.41	YIELD	-19.23	426.56	462.40	64.87	102.77	638.42
52	S-7	0.40	1.42	YIELD	-14.66	436.00	436.00	53.16	87.19	608.25
(Kabeyasawa and Hiraishi 1998)										
53	W-08	0.33	2.07	YIELD	-16.99	500.00	500.00	60.11	42.41	1030.28
54	W-12	0.29	2.75	YIELD	-20.65	500.00	500.00	61.24	46.66	1231.14
55	No. 1	0.42	1.30	YIELD	-11.82	500.00	500.00	65.24	50.05	649.04
56	No. 2	0.40	1.42	YIELD	-13.42	500.00	500.00	62.53	51.47	779.84
57	No. 3	0.40	1.44	YIELD	-15.29	500.00	500.00	60.31	51.71	915.32
58	No. 4	0.33	2.07	YIELD	-21.62	500.00	500.00	63.45	58.39	1195.28
59	No. 5	0.38	1.53	YIELD	-15.39	500.00	500.00	60.12	52.86	930.11
60	No. 6	0.39	1.48	YIELD	-16.56	500.00	500.00	59.09	52.25	1006.49
61	No. 7	0.40	1.43	YIELD	-19.95	500.00	500.00	56.74	51.63	1228.25
62	No. 8	0.38	1.52	YIELD	-24.59	500.00	500.00	54.52	52.72	1533.61
(Farvashany et al. 2008)										
63	HSCW1	0.33	2.08	YIELD	-13.98	428.00	428.00	59.69	106.65	521.35
64	HSCW2	0.35	1.86	YIELD	-17.21	428.00	428.00	63.22	102.75	557.31
65	HSCW3	0.36	1.72	YIELD	-14.88	428.00	428.00	61.70	100.10	510.74
66	HSCW4	0.35	1.82	YIELD	-26.74	428.00	428.00	68.51	102.01	678.76
67	HSCW5	0.37	1.68	YIELD	-18.24	428.00	428.00	60.30	99.32	607.29
68	HSCW6	0.35	1.80	YIELD	-14.99	428.00	428.00	56.89	101.63	556.73
69	HSCW7	0.33	2.04	YIELD	-16.39	428.00	428.00	57.75	105.96	598.89

Table B.3 Calculation of shear strengths of RC walls using the simplified truss model (continued).

No.	Specimen ID	ζ	σ_r (MPa)	Check	σ_d (MPa)	f_v (MPa)	f_h (MPa)	θ (deg)	D_u (kN)	V_n (kN)
(Burgueno et al. 2014)										
70	M05C	0.49	0.92	YIELD	-16.12	356.00	356.00	48.66	74.03	563.47
71	M05M	0.52	0.78	YIELD	-15.98	356.00	356.00	48.72	70.01	551.15
72	M10C	0.45	1.13	YIELD	-17.15	380.80	380.80	48.27	79.24	604.99
73	M10M	0.37	1.68	YIELD	-17.70	380.80	380.80	48.08	90.49	648.41
74	M15C	0.33	2.04	YIELD	-18.19	384.80	384.80	47.93	94.94	677.72
75	M15M	0.32	2.22	YIELD	-18.29	382.40	382.40	47.91	99.30	690.22
76	M20C	0.29	2.62	YIELD	-19.77	350.40	350.40	45.07	104.93	753.39
77	M20M	0.31	2.30	YIELD	-19.45	350.40	350.40	45.07	100.48	730.40
Author										
78	J1	0.33	2.07	YIELD	-9.96	488.00	488.00	57.71	186.66	664.70
79	J2	0.34	1.94	YIELD	-11.61	462.40	488.00	60.41	182.66	694.50
80	J3	0.32	2.21	YIELD	-12.58	488.00	462.40	51.71	191.02	824.34
81	J4	0.35	1.87	YIELD	-9.28	488.00	488.00	57.40	26.57	390.89
82	J5	0.33	2.07	YIELD	-9.96	488.00	488.00	57.71	186.66	664.70
83	J6	0.34	1.94	YIELD	-11.61	462.40	488.00	60.41	182.66	694.50
84	J7	0.32	2.21	YIELD	-12.58	488.00	462.40	51.71	191.02	824.34

Table B.4 Calculation of shear strengths of RC walls using the proposed strut-tie model.

No.	Specimen ID	c (mm)	r (mm)	θ (deg)	A_{str} (mm ²)	ζ	D_n (kN)	V_n (kN)
(Barda et al. 1977)								
1	B1-1	999.44	175.20	37.75	62169.28	0.56	1011.65	799.88
2	B2-1	1228.97	76.83	38.12	77087.91	0.71	893.64	703.00
3	B3-2	1114.21	111.55	37.61	69087.99	0.58	1081.29	856.57
4	B6-4	1114.21	74.83	36.80	67817.22	0.57	821.74	657.96
5	B7-5	1524.76	60.69	31.00	79787.67	0.58	1197.30	1026.29
6	B8-5	830.18	144.10	54.76	68888.89	0.60	974.52	562.30
(Cardenas et al. 1980)								
7	SW-7	754.38	288.04	58.93	49235.55	0.46	978.56	505.06
8	SW-8	701.04	601.98	59.00	45789.23	0.59	1148.66	591.60
(Corley et al. 1981)								
9	B2	614.79	239.06	59.00	53540.80	0.44	1272.41	655.34
10	B5	615.37	230.28	59.00	53591.52	0.47	1140.73	587.52
11	B6	767.35	480.20	59.00	66827.17	0.62	902.37	464.75
12	B7	698.78	517.79	59.00	60855.92	0.45	1359.85	700.37
13	B8	714.10	520.99	59.00	62189.56	0.55	1448.50	746.03
14	B9	709.14	529.28	59.00	61757.30	0.47	1275.93	657.15
15	B10	640.18	645.40	59.00	55752.25	0.46	1177.63	606.52
16	F1	624.54	140.23	59.00	54389.99	0.51	1076.59	554.49
17	F2	732.32	436.86	59.00	63776.64	0.46	1348.91	694.74
(Maeda 1986)								
18	MAE03	1265.87	407.78	46.49	73442.39	0.46	1950.91	1343.24
19	MAE07	1266.06	419.70	46.79	73821.50	0.52	2212.17	1514.62
(Okamoto 1990)								
20	W48M6	766.98	540.28	58.12	78151.05	0.38	2464.57	1301.78
21	W48M4	783.03	564.48	59.00	80542.37	0.38	2509.97	1292.73
22	W72M8	766.98	494.95	56.68	76904.78	0.42	2673.52	1468.75

Table B.4 Calculation of shear strengths of RC walls using the proposed strut-tie model (continued).

No.	Specimen ID	c (mm)	r (mm)	θ (deg)	A_{str} (mm ²)	ζ	D_n (kN)	V_n (kN)
23	W72M6	783.03	499.25	57.06	78860.80	0.41	2688.00	1461.50
24	W72M8	759.07	496.44	56.60	76043.99	0.36	2758.29	1518.41
25	W96M8	775.12	458.47	55.68	76815.80	0.38	2956.97	1667.39
(Mo and Chan 1996)								
26	HN4-1	404.27	131.50	51.00	21992.13	0.53	373.74	235.20
27	HN4-2	404.27	131.50	51.00	21992.13	0.53	373.74	235.20
28	HN4-3	404.27	131.50	51.00	21992.13	0.53	372.99	234.73
29	HN6-1	404.27	131.50	51.00	21992.13	0.60	389.85	245.35
30	HN6-2	404.27	131.50	51.00	21992.13	0.60	389.85	245.35
31	HN6-3	404.27	131.50	51.00	21992.13	0.59	401.07	252.41
32	HM4-1	404.27	131.50	51.00	21992.13	0.50	412.20	259.41
33	HM4-2	404.27	131.50	51.00	21992.13	0.50	412.20	259.41
34	HM4-3	404.27	131.50	51.00	21992.13	0.49	428.84	269.88
35	LN4-1	404.27	119.59	50.37	21795.52	0.64	249.75	159.30
36	LN4-2	404.27	119.59	50.37	21795.52	0.64	249.75	159.30
37	LN4-3	404.27	119.59	50.37	21795.52	0.53	345.02	220.06
38	LN6-1	404.27	119.59	50.37	21795.52	0.58	385.67	245.99
39	LN6-2	404.27	119.59	50.37	21795.52	0.58	381.95	243.62
40	LN6-3	404.27	119.59	50.37	21795.52	0.58	381.95	243.62
41	LM6-1	404.27	119.59	50.37	21795.52	0.52	446.38	284.71
42	LM6-2	404.27	119.59	50.37	21795.52	0.53	430.75	274.74
43	LM6-3	404.27	119.59	50.37	21795.52	0.55	413.26	263.58
44	LM4-2	404.27	119.59	50.37	21795.52	0.39	567.68	362.07
45	LM4-3	404.27	119.59	50.37	21795.52	0.39	567.68	362.07
(Gupta and Rangan 1998)								
46	S-1	412.91	144.29	59.00	26545.10	0.38	807.60	415.94
47	S-2	503.54	277.88	59.00	32371.35	0.43	903.59	465.39

Table B.4 Calculation of shear strengths of RC walls using the proposed strut-tie model (continued).

No.	Specimen ID	c (mm)	r (mm)	θ (deg)	A_{str} (mm ²)	ζ	D_n (kN)	V_n (kN)
48	S-3	585.03	321.66	59.00	37610.31	0.42	1078.66	555.55
49	S-4	444.46	133.87	59.00	28573.50	0.41	883.09	454.82
50	S-5	525.22	250.59	59.00	33764.82	0.42	1031.98	531.51
51	S-6	599.98	305.87	59.00	38570.99	0.43	1161.64	598.29
52	S-7	498.19	278.23	59.00	32027.29	0.44	993.33	511.60
(Kabeyasawa and Hiraishi 1998)								
53	W-08	717.69	640.45	59.00	49214.61	0.34	1712.17	881.83
54	W-12	716.20	675.12	59.00	49112.34	0.25	1673.09	861.70
55	No. 1	841.42	427.37	59.00	57699.14	0.39	1447.46	745.50
56	No. 2	830.07	429.33	59.00	56920.37	0.39	1576.08	811.74
57	No. 3	828.26	431.56	59.00	56796.49	0.41	1683.60	867.12
58	No. 4	848.59	520.77	59.00	58190.49	0.32	1936.62	997.43
59	No. 5	697.31	437.31	59.00	47816.67	0.40	1456.40	750.10
60	No. 6	824.29	438.80	59.00	56524.26	0.49	2038.98	1050.15
61	No. 7	828.80	436.65	59.00	56833.29	0.48	1934.58	996.38
62	No. 8	821.03	441.89	59.00	56300.92	0.51	2191.27	1128.59
(Farvashany et al. 2008)								
63	HSCW1	412.14	202.74	59.00	26495.30	0.32	879.42	452.93
64	HSCW2	443.03	249.12	59.00	28481.01	0.35	922.11	474.92
65	HSCW3	448.05	248.36	59.00	28803.75	0.35	878.02	452.22
66	HSCW4	538.88	328.09	59.00	34643.03	0.34	1076.71	554.54
67	HSCW5	449.80	248.90	59.00	28916.50	0.39	937.23	482.71
68	HSCW6	417.74	203.73	59.00	26855.46	0.37	888.47	457.59
69	HSCW7	437.39	248.57	59.00	28118.53	0.32	927.62	477.76
(Burgueno et al. 2014)								
70	M05C	521.70	228.71	59.00	33986.22	0.62	965.03	497.03
71	M05M	532.63	228.14	59.00	34698.07	0.68	919.68	473.67

Table B.4 Calculation of shear strengths of RC walls using the proposed strut-tie model (continued).

No.	Specimen ID	c (mm)	r (mm)	θ (deg)	A_{str} (mm ²)	ζ	D_n (kN)	V_n (kN)
72	M10C	510.66	235.15	59.00	33267.04	0.56	1047.88	539.70
73	M10M	494.62	236.16	59.00	32221.86	0.43	1153.71	594.20
74	M15C	478.52	244.45	59.00	31173.26	0.36	1157.22	596.01
75	M15M	486.64	227.99	59.00	31702.34	0.34	1179.93	607.71
76	M20C	482.86	237.52	59.00	31455.57	0.28	1171.05	603.14
77	M20M	485.78	237.34	59.00	31646.12	0.33	1205.31	620.78
Author								
78	J1	538.68	240.85	59.00	46173.89	0.30	1421.50	732.13
79	J2	538.68	236.16	59.00	46173.89	0.32	1443.25	743.33
80	J3	538.68	248.05	59.00	46173.89	0.30	1512.37	778.93
81	J4	517.51	236.71	59.00	44359.16	0.32	1326.24	683.06
82	J5	408.24	243.34	59.00	34993.26	0.30	1077.30	554.85
83	J6	408.24	242.42	59.00	34993.26	0.32	1093.78	563.34
84	J7	408.24	250.41	59.00	34993.26	0.30	1146.16	590.32

APPENDIX C

SUBROUTINES FOR CALCULATION OF SHEAR STRENGTHS OF RC WALLS

In this part, subroutines written in MACRO VBA-script for Microsoft® Excel which are used to calculate shear strengths of RC walls using various methods are presented.

C.1 ACI 318 (2014)

Sub ACI()

'

' ACI 318 Chapter 18 Shear Strength of RC Wall

'

Dim i As Integer

For i = 3 To 176

 'Defining input variables

 fc = Worksheets("All").Cells(i, "D").Value 'concrete cylinder compressive strength (in MPa)

 Ag = Worksheets("All").Cells(i, "E").Value 'gross area of wall cross section (in mm²)

 P = Worksheets("All").Cells(i, "F").Value 'axial load applied (in kN)

 Hw = Worksheets("All").Cells(i, "J").Value 'clear height of wall (in mm)

 Lw = Worksheets("All").Cells(i, "K").Value 'length of wall (in mm)

 dw = Worksheets("All").Cells(i, "L").Value 'effective depth of wall (in mm)

 tw = Worksheets("All").Cells(i, "M").Value 'thickness of wall web (in mm)

 bf = Worksheets("All").Cells(i, "N").Value 'boundary element width (in mm)

 tf = Worksheets("All").Cells(i, "O").Value 'boundary element thickness (in mm)

 Hprime = Worksheets("All").Cells(i, "P").Value 'distance of point of application of shear force to wall base (in mm)

pb = Worksheets("All").Cells(i, "Q").Value 'boundary element vertical reinforcement ratio

fyb = Worksheets("All").Cells(i, "R").Value 'boundary element vertical reinforcement yield strength (in MPa)

pwv = Worksheets("All").Cells(i, "W").Value 'wall web vertical reinforcement ratio

fywv = Worksheets("All").Cells(i, "X").Value 'wall web vertical reinforcement yield strength (in MPa)

pwh = Worksheets("All").Cells(i, "Y").Value 'wall web horizontal reinforcement ratio

fywh = Worksheets("All").Cells(i, "Z").Value 'wall web horizontal reinforcement yield strength (in MPa)

Vexp = Worksheets("All").Cells(i, "AA").Value 'experimental wall strength (in kN)

drift = Worksheets("All").Cells(i, "AB").Value 'drift ratio at experimental wall strength

behavior = Worksheets("All").Cells(i, "AC").Value 'failure mode

'Calculating shear strength of RC wall based on ACI 318 chapter 18

'Defining alphac

If Hw / Lw <= 1.5 Then

 alphac = 0.25

ElseIf Hw / Lw >= 2 Then

 alphac = 0.17

Else

 alphac = 0.25 - 0.08 * (Hw / Lw - 1.5) / (2 - 1.5)

End If

Vnaci = Lw * tw * (alphac * fc ^ 0.5 + pwh * fywh) / 1000 'nominal shear strength of wall in kN

Vnacimax = 0.83 * Lw * tw * fc ^ 0.5 / 1000 'maximum shear strength limit in kN

```
If Vnaci < Vnacimax Then
    Vaci = Vnaci
Else
    Vaci = Vnacimax
End If

If behavior = "SHEAR" Then
    Worksheets("All").Cells(i, "QA").Value = Vaci
    Worksheets("All").Cells(i, "QB").Value = Vexp / Vaci
End If

Next i

'
End Sub
```

C.2 Eurocode 8 (2004)

```
Sub EC()
```

```
'
```

```
' Eurocode 8 Shear Strength of RC Wall
```

```
'
```

```
Dim i As Integer
```

```
For i = 3 To 176
```

```
    'Defining input variables
```

```
    fc = Worksheets("All").Cells(i, "D").Value 'concrete cylinder compressive
    strength (in MPa)
```

```
    Ag = Worksheets("All").Cells(i, "E").Value 'gross area of wall cross section
    (in mm2)
```

$P = \text{Worksheets("All").Cells}(i, "F").\text{Value}$ 'axial load applied (in kN)
 $H_w = \text{Worksheets("All").Cells}(i, "J").\text{Value}$ 'clear height of wall (in mm)
 $L_w = \text{Worksheets("All").Cells}(i, "K").\text{Value}$ 'length of wall (in mm)
 $d_w = \text{Worksheets("All").Cells}(i, "L").\text{Value}$ 'effective depth of wall (in mm)
 $t_w = \text{Worksheets("All").Cells}(i, "M").\text{Value}$ 'thickness of wall web (in mm)
 $b_f = \text{Worksheets("All").Cells}(i, "N").\text{Value}$ 'boundary element width (in mm)
 $t_f = \text{Worksheets("All").Cells}(i, "O").\text{Value}$ 'boundary element thickness (in mm)
 $H_{\text{prime}} = \text{Worksheets("All").Cells}(i, "P").\text{Value}$ 'distance of point of application of shear force to wall base (in mm)
 $p_b = \text{Worksheets("All").Cells}(i, "Q").\text{Value}$ 'boundary element vertical reinforcement ratio
 $f_{yb} = \text{Worksheets("All").Cells}(i, "R").\text{Value}$ 'boundary element vertical reinforcement yield strength (in MPa)
 $p_{wv} = \text{Worksheets("All").Cells}(i, "W").\text{Value}$ 'wall web vertical reinforcement ratio
 $f_{ywv} = \text{Worksheets("All").Cells}(i, "X").\text{Value}$ 'wall web vertical reinforcement yield strength (in MPa)
 $p_{wh} = \text{Worksheets("All").Cells}(i, "Y").\text{Value}$ 'wall web horizontal reinforcement ratio
 $f_{ywh} = \text{Worksheets("All").Cells}(i, "Z").\text{Value}$ 'wall web horizontal reinforcement yield strength (in MPa)
 $V_{\text{exp}} = \text{Worksheets("All").Cells}(i, "AA").\text{Value}$ 'experimental wall strength (in kN)
 $\text{drift} = \text{Worksheets("All").Cells}(i, "AB").\text{Value}$ 'drift ratio at experimental wall strength
 $\text{behavior} = \text{Worksheets("All").Cells}(i, "AC").\text{Value}$ 'failure mode

'Calculating shear strength of RC wall based on Eurocode 8
'Defining VR_{dc}
'Defining CR_{dc}
 $CR_{dc} = 0.18$

```

'Defining k
If  $1 + (200 / (Lw - 0.5 * tf)) ^ 0.5 \leq 2$  Then
     $k = 1 + (200 / (Lw - 0.5 * tf)) ^ 0.5$ 
Else
     $k = 2$ 
End If

'Defining Rhol
If  $(pb * bf * tf + 0.5 * pwv * tw * (Lw - 2 * tf)) / (tw * (Lw - 0.5 * tf)) \leq 0.02$ 
Then
     $Rhol = (pb * bf * tf + 0.5 * pwv * tw * (Lw - 2 * tf)) / (tw * (Lw - 0.5 * tf))$ 
Else
     $Rhol = 0.02$ 
End If

'Defining k1
 $k1 = 0.15$ 

'Defining sigmacp
If  $P * 1000 / Ag < 0.2 * fc$  Then
     $sigmacp = P * 1000 / Ag$ 
Else
     $sigmacp = 0.2 * fc$ 
End If

'Defining vmin
 $vmin = 0.035 * k ^ 1.5 * fc ^ 0.5$ 
If  $CRdc * k * (100 * Rhol * fc) ^ (1 / 3) + k1 * sigmacp > vmin + k1 * sigmacp$ 
Then
     $VRdc = (CRdc * k * (100 * Rhol * fc) ^ (1 / 3) + k1 * sigmacp) * tw * (Lw - 0.5 * tf) / 1000$ 
Else
     $VRdc = (vmin + k1 * sigmacp) * tw * (Lw - 0.5 * tf) / 1000$ 
End If

'Defining VRds

```


'Defining cottheta and tantheta

cottheta = 1

tantheta = 1

$VRds = pwh * tw * 0.8 * Lw * fywh * cottheta / 1000$

'Defining VRdmax

'Defining alphacw

If $P * 1000 / Ag = 0$ Then

 alphacw = 1

ElseIf $P * 1000 / Ag \leq 0.25 * fc$ Then

 alphacw = $1 + P * 1000 / Ag / fc$

ElseIf $P * 1000 / Ag \leq 0.5 * fc$ Then

 alphacw = 1.25

Else

 alphacw = $2.5 * (1 - P * 1000 / Ag / fc)$

End If

'Defining v1

$v1 = 0.6 * (1 - fc / 250)$

$VRdmax = alphacw * tw * 0.8 * Lw * v1 * fc / (cottheta + tantheta) / 1000$

'Diagonal compression failure (VRdcomp)

$VRdcomp = 0.4 * VRdmax$

'Diagonal tension failure (VRdtens)

$alphas = Hprime / Lw$

If $alphas \geq 2$ Then

 VRdtens = VRds

Else

 VRdtens = $VRdc + 0.75 * pwh * fywh * tw * alphas * Lw / 1000$

End If

If $VRdcomp < VRdtens$ Then

```
        Vec = VRdcomp
    Else
        Vec = VRdtens
    End If

    If behavior = "SHEAR" Then
        Worksheets("All").Cells(i, "QC").Value = Vec
        Worksheets("All").Cells(i, "QD").Value = Vexp / Vec
    End If

Next i

'
End Sub
```

C.3 Hwang and Lee (2002)

```
Sub HwangLee()
'
' Hwang and Lee (2002) Shear Strength of RC Wall
'

    Dim i As Integer

    For i = 3 To 176

        'Defining input variables
        fc = Worksheets("All").Cells(i, "D").Value 'concrete cylinder compressive
        strength (in MPa)
        Ag = Worksheets("All").Cells(i, "E").Value 'gross area of wall cross section
        (in mm2)
        P = Worksheets("All").Cells(i, "F").Value 'axial load applied (in kN)
        Hw = Worksheets("All").Cells(i, "J").Value 'clear height of wall (in mm)
        Lw = Worksheets("All").Cells(i, "K").Value 'length of wall (in mm)
```

```

dw = Worksheets("All").Cells(i, "L").Value 'effective depth of wall (in mm)
tw = Worksheets("All").Cells(i, "M").Value 'thickness of wall web (in mm)
bf = Worksheets("All").Cells(i, "N").Value 'boundary element width (in mm)
tf = Worksheets("All").Cells(i, "O").Value 'boundary element thickness (in
mm)
Hprime = Worksheets("All").Cells(i, "P").Value 'distance of point of
application of shear force to wall base (in mm)
pb = Worksheets("All").Cells(i, "Q").Value 'boundary element vertical
reinforcement ratio
fyb = Worksheets("All").Cells(i, "R").Value 'boundary element vertical
reinforcement yield strength (in MPa)
pwv = Worksheets("All").Cells(i, "W").Value 'wall web vertical reinforcement
ratio
fywv = Worksheets("All").Cells(i, "X").Value 'wall web vertical reinforcement
yield strength (in MPa)
pwh = Worksheets("All").Cells(i, "Y").Value 'wall web horizontal
reinforcement ratio
fywh = Worksheets("All").Cells(i, "Z").Value 'wall web horizontal
reinforcement yield strength (in MPa)
Vexp = Worksheets("All").Cells(i, "AA").Value 'experimental wall strength
(in kN)
drift = Worksheets("All").Cells(i, "AB").Value 'drift ratio at experimental wall
strength
behavior = Worksheets("All").Cells(i, "AC").Value 'failure mode

'Calculating shear strength of RC wall based on Hwang and Lee (2002)
'Defining area of strut
Astr = tw * (0.25 + 0.85 * (P * 1000) / (tw * Lw * fc)) * Lw
'Defining zheta
zheta = Application.WorksheetFunction.Min(3.35 / (fc ^ 0.5), 0.52)
'Defining theta
theta = Atn(Hprime / dw)

```

```

'Determining strut and tie index (k)
'Determining gammah and gammav
If (2 * Tan(theta) - 1) / 3 <= 0 Then
    gammah = 0
ElseIf (2 * Tan(theta) - 1) / 3 >= 1 Then
    gammah = 1
Else
    gammah = (2 * Tan(theta) - 1) / 3
End If
If (2 * 1 / Tan(theta) - 1) / 3 <= 0 Then
    gammav = 0
ElseIf (2 * 1 / Tan(theta) - 1) / 3 >= 1 Then
    gammav = 1
Else
    gammav = (2 * 1 / Tan(theta) - 1) / 3
End If
'Determining Khbar and Kvbar
Khbar = 1 / (1 - 0.2 * (gammah + gammah ^ 2))
Kvbar = 1 / (1 - 0.2 * (gammav + gammav ^ 2))
'Determining Fhbar and Fvbar
Fhbar = gammah * Khbar * zheta * fc * Astr * Cos(theta)
Fvbar = gammav * Kvbar * zheta * fc * Astr * Sin(theta)
'Determining fyh and fyv
fyh = pwh * fywh * Hw * tw
fyv = pwv * fywv * (Lw - 2 * tf) * tw
'Determining Kh and Kv
If gammah = 0 Then
    Kh = 1
Else
    Kh = Application.WorksheetFunction.Min(1 + (Khbar - 1) * fyh / Fhbar,
    Khbar)

```

```

End If
If gammav = 0 Then
    Kv = 1
Else
    Kv = Application.WorksheetFunction.Min(1 + (Kvbar - 1) * fyv / Fvbar,
Kvbar)
End If
k = Kh + Kv - 1

'Determining shear strength of RC Wall
Vsst = k * zheta * fc * Astr * Cos(theta) / 1000

If behavior = "SHEAR" Then
    Worksheets("All").Cells(i, "QE").Value = Vsst
    Worksheets("All").Cells(i, "QF").Value = Vexp / Vsst
End If

Next i

'
End Sub

```

C.4 Gupta and Rangan (1998)

```

Sub GuptaRangan()
'
' Gupta and Rangan (1998) Shear Strength of RC Wall
'

Dim i As Integer

For i = 3 To 176

```

'Defining input variables

fc = Worksheets("All").Cells(i, "D").Value 'concrete cylinder compressive strength (in MPa)

Ag = Worksheets("All").Cells(i, "E").Value 'gross area of wall cross section (in mm²)

P = Worksheets("All").Cells(i, "F").Value 'axial load applied (in kN)

Hw = Worksheets("All").Cells(i, "J").Value 'clear height of wall (in mm)

Lw = Worksheets("All").Cells(i, "K").Value 'length of wall (in mm)

dw = Worksheets("All").Cells(i, "L").Value 'effective depth of wall (in mm)

tw = Worksheets("All").Cells(i, "M").Value 'thickness of wall web (in mm)

bf = Worksheets("All").Cells(i, "N").Value 'boundary element width (in mm)

tf = Worksheets("All").Cells(i, "O").Value 'boundary element thickness (in mm)

Hprime = Worksheets("All").Cells(i, "P").Value 'distance of point of application of shear force to wall base (in mm)

pb = Worksheets("All").Cells(i, "Q").Value 'boundary element vertical reinforcement ratio

fyb = Worksheets("All").Cells(i, "R").Value 'boundary element vertical reinforcement yield strength (in MPa)

pwv = Worksheets("All").Cells(i, "W").Value 'wall web vertical reinforcement ratio

fywv = Worksheets("All").Cells(i, "X").Value 'wall web vertical reinforcement yield strength (in MPa)

pwh = Worksheets("All").Cells(i, "Y").Value 'wall web horizontal reinforcement ratio

fywh = Worksheets("All").Cells(i, "Z").Value 'wall web horizontal reinforcement yield strength (in MPa)

Vexp = Worksheets("All").Cells(i, "AA").Value 'experimental wall strength (in kN)

drift = Worksheets("All").Cells(i, "AB").Value 'drift ratio at experimental wall strength

behavior = Worksheets("All").Cells(i, "AC").Value 'failure mode

```

'Calculating shear strength of RC wall based on Gupta and Rangan (1998)
alpha = Atn(dw / Hw)
epsilono = 0.002 + 0.001 * (fc - 20) / 80
Esteel = 200000
Econcrete = 3320 * (fc ^ 0.5) + 6900
epsilon_r = Worksheets("All").Cells(i, "GX").Value
zeta = (1 / (1 + 400 * epsilon_r) ^ 0.5) *
Application.WorksheetFunction.Min(5.8 / (fc ^ 0.5), 0.9)
sigma_d = -zeta * fc
epsilon_d = -zeta * epsilono
If epsilon_r <= 0.00008 Then
    sigma_r = Econcrete * epsilon_r
Else
    sigma_r = Econcrete * 0.00008 * (0.00008 / epsilon_r) ^ 0.4
End If
epsilon_v = epsilon_d * (Cos(alpha)) ^ 2 + epsilon_r * (Sin(alpha)) ^ 2
If epsilon_v >= 0 Then
    fwv = Application.WorksheetFunction.Min(fywv, Esteel * epsilon_v)
Else
    fwv = Application.WorksheetFunction.Max(-fywv, Esteel * epsilon_v)
End If

sigma_v = sigma_d * (Cos(alpha)) ^ 2 + sigma_r * (Sin(alpha)) ^ 2 + pwv * fwv

If Abs(-P * 1000 / Ag - sigma_v) < 0.000001 Then
    Vstm = (sigma_r - sigma_d) * Sin(alpha) * Cos(alpha) * tw * Lw / 1000
Else
    Vstm = 0
End If

If behavior = "SHEAR" Then

```

```

Worksheets("All").Cells(i, "QG").Value = Vstm
Worksheets("All").Cells(i, "QH").Value = Vexp / Vstm
End If

Next i

,

End Sub

```

C.5 Complete Truss Model

```

Sub CompleteTrussModel()
,
' Complete Truss Model for Shear Strength of RC Wall
,

Dim i As Integer

For i = 3 To 10000
If Cells(i, "TA").Value = 0 Then
    GoTo 1000
End If

'Defining input variables
fc = Cells(i, "D").Value 'concrete cylinder compressive strength (in MPa)
Ag = Cells(i, "E").Value 'gross area of wall cross section (in mm2)
P = Cells(i, "F").Value 'axial load applied (in kN)
Hw = Cells(i, "J").Value 'clear height of wall (in mm)
Lw = Cells(i, "K").Value 'length of wall (in mm)
dw = Cells(i, "L").Value 'effective depth of wall (in mm)
tw = Cells(i, "M").Value 'thickness of wall web (in mm)
bf = Cells(i, "N").Value 'boundary element width (in mm)
tf = Cells(i, "O").Value 'boundary element thickness (in mm)

```


Hprime = Cells(i, "P").Value 'distance of point of application of shear force to wall base (in mm)

pb = Cells(i, "Q").Value 'boundary element vertical reinforcement ratio

fyb = Cells(i, "R").Value 'boundary element vertical reinforcement yield strength (in MPa)

pwv = Cells(i, "W").Value 'wall web vertical reinforcement ratio

fywv = Cells(i, "X").Value 'wall web vertical reinforcement yield strength (in MPa)

pwh = Cells(i, "Y").Value 'wall web horizontal reinforcement ratio

fywh = Cells(i, "Z").Value 'wall web horizontal reinforcement yield strength (in MPa)

Vexp = Cells(i, "AA").Value 'experimental wall strength (in kN)

drift = Cells(i, "AB").Value 'drift ratio at experimental wall strength

behavior = Cells(i, "AC").Value 'failure mode

'Defining known variables

sigmav = -P * 1000 / Ag

sigmah = 0

'Taking the values of epsilon_d and epsilon_r

epsilon_d = Cells(i, "TA").Value

epsilon_r = Cells(i, "TB").Value

'Calculating sigma_r and sigma_d

Econcrete = (3320 * (fc) ^ 0.5 + 6900)

Esteel = 200000

epsilon_{no} = 0.002 + 0.001 * (fc - 20) / 80

100

If epsilon_r <= 0.00008 Then

 sigma_r = Econcrete * epsilon_r

Else

$$\text{sigmar} = 0.02 * \text{fc}$$

End If

If $(5.8 / (\text{fc} ^ 0.5)) * (1 / (1 + 400 * \text{epsilon}_{nr}) ^ 0.5) \leq 0.9 * (1 / (1 + 400 * \text{epsilon}_{nr}) ^ 0.5)$ Then

$$\text{zheta} = (5.8 / (\text{fc} ^ 0.5)) * (1 / (1 + 400 * \text{epsilon}_{nr}) ^ 0.5)$$

Else

$$\text{zheta} = 0.9 * (1 / (1 + 400 * \text{epsilon}_{nr}) ^ 0.5)$$

End If

If $\text{epsilon}_{nd} \geq -\text{zheta} * \text{epsilon}_{no}$ Then

$$\text{sigmad} = -\text{zheta} * \text{fc} * (2 * (\text{epsilon}_{nd} / (-\text{zheta} * \text{epsilon}_{no})) - (\text{epsilon}_{nd} / (-\text{zheta} * \text{epsilon}_{no})) ^ 2)$$

Else

$$\text{sigmad} = -\text{zheta} * \text{fc} * (1 - ((\text{epsilon}_{nd} / (-\text{zheta} * \text{epsilon}_{no}) - 1) / (2 / \text{zheta} - 1)) ^ 2)$$

End If

'Calculating epsilon_{nv} and epsilon_{nh}

$$\text{fvmax} = \text{Application.WorksheetFunction.Min}(0.8 * \text{fy}_{wv}, 500)$$

$$\text{fhmax} = \text{Application.WorksheetFunction.Min}(0.8 * \text{fy}_{wh}, 500)$$

$$\text{epsilon}_{nv} = (\text{epsilon}_{nr} * (-\text{sigmad} + \text{sigmav}) + \text{epsilon}_{nd} * (-\text{sigmav} + \text{sigmar})) / ((-\text{sigmad} + \text{sigmar}) - \text{pwv} * \text{E}_{steel} * (\text{epsilon}_{nd} - \text{epsilon}_{nr}))$$

If $\text{epsilon}_{nv} \leq -\text{fvmax} / \text{E}_{steel}$ Then

$$\text{epsilon}_{nv} = ((\text{epsilon}_{nd} - \text{epsilon}_{nr}) * (-\text{sigmav} + \text{pwv} * -\text{fvmax}) - \text{sigmad} * \text{epsilon}_{nr} + \text{sigmar} * \text{epsilon}_{nd}) / (-\text{sigmad} + \text{sigmar})$$

ElseIf $\text{epsilon}_{nv} \geq \text{fvmax} / \text{E}_{steel}$ Then

$$\text{epsilon}_{nv} = ((\text{epsilon}_{nd} - \text{epsilon}_{nr}) * (-\text{sigmav} + \text{pwv} * \text{fvmax}) - \text{sigmad} * \text{epsilon}_{nr} + \text{sigmar} * \text{epsilon}_{nd}) / (-\text{sigmad} + \text{sigmar})$$

End If

$\epsilon_{nh} = \epsilon_{nd} + \epsilon_{nr} - \epsilon_{nv}$

'Calculating f_v and f_h

$f_{wv} = E_{steel} * \epsilon_{nv}$

If $f_{wv} < -Application.WorksheetFunction.Min(0.8 * f_{ywv}, 500)$ Then

$f_{wv} = -Application.WorksheetFunction.Min(0.8 * f_{ywv}, 500)$

ElseIf $f_{wv} > Application.WorksheetFunction.Min(0.8 * f_{ywv}, 500)$ Then

$f_{wv} = Application.WorksheetFunction.Min(0.8 * f_{ywv}, 500)$

End If

$f_{wh} = E_{steel} * \epsilon_{nh}$

If $f_{wh} < -Application.WorksheetFunction.Min(0.8 * f_{ywh}, 500)$ Then

$f_{wh} = -Application.WorksheetFunction.Min(0.8 * f_{ywh}, 500)$

ElseIf $f_{wh} > Application.WorksheetFunction.Min(0.8 * f_{ywh}, 500)$ Then

$f_{wh} = Application.WorksheetFunction.Min(0.8 * f_{ywh}, 500)$

End If

'Checking equilibriums

If $(-\sigma_{mv} - \sigma_{mh} + \sigma_{md} + \sigma_{mr} + p_{wv} * f_{wv} + p_{wh} * f_{wh}) > 0.01$

Then

$\epsilon_{nr} = \epsilon_{nr} + 0.000001$

GoTo 100

ElseIf $(-\sigma_{mv} - \sigma_{mh} + \sigma_{md} + \sigma_{mr} + p_{wv} * f_{wv} + p_{wh} * f_{wh}) < -0.01$ Then

$\epsilon_{nr} = \epsilon_{nr} + 0.000001$

GoTo 100

End If

'Calculating θ , τ_{vh} , γ_{mvh}

$\theta = Application.WorksheetFunction.Asin(((-\sigma_{mv} + \sigma_{mr} + p_{wv} * f_{wv}) / (-\sigma_{md} + \sigma_{mr})) ^ 0.5)$

$$\tau_{vh} = (-\sigma_{ad} + \sigma_{ar}) * \sin(\theta) * \cos(\theta)$$

$$\gamma_{mh} = 2 * (-\epsilon_{ld} + \epsilon_{lr}) * \sin(\theta) * \cos(\theta)$$

'Calculating dowel action

If $b_f \leq 0.5 * H_w + t_w$ Then

$$b_{fuse} = b_f$$

Else

$$b_{fuse} = 0.5 * H_w + t_w$$

End If

$$A_{sb} = p_b * b_{fuse} * t_f$$

$$d_{be} = (A_{sb} / (0.25 * \text{Application.WorksheetFunction.Pi()})) ^ {0.5}$$

$$D_u = 1.64 * (b_{fuse} - d_{be}) * d_{be} * (f_c ^ {1 / 3}) / 1000$$

'Calculating force and deformation of RC wall

$$V_{complete\ model} = \tau_{vh} * t_w * d_w / 1000 + D_u$$

$$\Delta w = \gamma_{mh} * H_{prime}$$

'Putting values into cells

$$\text{Cells}(i, "TC").\text{Value} = \epsilon_{lv}$$

$$\text{Cells}(i, "TD").\text{Value} = \epsilon_{lh}$$

$$\text{Cells}(i, "TE").\text{Value} = \sigma_{ad}$$

$$\text{Cells}(i, "TF").\text{Value} = \sigma_{ar}$$

$$\text{Cells}(i, "TG").\text{Value} = f_{wv}$$

$$\text{Cells}(i, "TH").\text{Value} = f_{wh}$$

$$\text{Cells}(i, "TI").\text{Value} = \theta$$

$$\text{Cells}(i, "TJ").\text{Value} = \tau_{vh}$$

$$\text{Cells}(i, "TK").\text{Value} = \gamma_{mh}$$

$$\text{Cells}(i, "TL").\text{Value} = V_{complete\ model}$$

$$\text{Cells}(i, "TM").\text{Value} = \Delta w$$

$$\text{Cells}(i, "TN").\text{Value} = V_{exp} / V_{complete\ model}$$

Next i

1000

End Sub

C.6 Simplified Truss Model

Sub SimplifiedTrussModel()

,

' Simplified Truss Model Shear Strength of RC Wall

,

Dim i As Integer

For i = 3 To 86

'Defining input variables

fc = Worksheets("All").Cells(i, "D").Value 'concrete cylinder compressive strength (in MPa)

Ag = Worksheets("All").Cells(i, "E").Value 'gross area of wall cross section (in mm²)

P = Worksheets("All").Cells(i, "F").Value 'axial load applied (in kN)

Hw = Worksheets("All").Cells(i, "J").Value 'clear height of wall (in mm)

Lw = Worksheets("All").Cells(i, "K").Value 'length of wall (in mm)

dw = Worksheets("All").Cells(i, "L").Value 'effective depth of wall (in mm)

tw = Worksheets("All").Cells(i, "M").Value 'thickness of wall web (in mm)

bf = Worksheets("All").Cells(i, "N").Value 'boundary element width (in mm)

tf = Worksheets("All").Cells(i, "O").Value 'boundary element thickness (in mm)

Hprime = Worksheets("All").Cells(i, "P").Value 'distance of point of application of shear force to wall base (in mm)

pb = Worksheets("All").Cells(i, "Q").Value 'boundary element vertical reinforcement ratio

fyb = Worksheets("All").Cells(i, "R").Value 'boundary element vertical reinforcement yield strength (in MPa)

pwv = Worksheets("All").Cells(i, "W").Value 'wall web vertical reinforcement ratio

fywv = Worksheets("All").Cells(i, "X").Value 'wall web vertical reinforcement yield strength (in MPa)

pwh = Worksheets("All").Cells(i, "Y").Value 'wall web horizontal reinforcement ratio

fywh = Worksheets("All").Cells(i, "Z").Value 'wall web horizontal reinforcement yield strength (in MPa)

Vexp = Worksheets("All").Cells(i, "AA").Value 'experimental wall strength (in kN)

drift = Worksheets("All").Cells(i, "AB").Value 'drift ratio at experimental wall strength

behavior = Worksheets("All").Cells(i, "AC").Value 'failure mode

'Calculating shear strength of RC wall based on Proposed Model

epsilon_r = 0.005

sigma_v = -P * 1000 / A_g

sigma_h = 0

'Defining sigma_r

f_{cr} = 0.31 * (f_c ^ 0.5)

sigma_r = 0.02 * f_c

'Defining zeta

If (5.8 / (f_c ^ 0.5)) * (1 / (1 + 400 * epsilon_r) ^ 0.5) <= 0.9 * (1 / (1 + 400 * epsilon_r) ^ 0.5) Then

zeta = (5.8 / (f_c ^ 0.5)) * (1 / (1 + 400 * epsilon_r) ^ 0.5)

Else

zeta = 0.9 * (1 / (1 + 400 * epsilon_r) ^ 0.5)

End If

'Check whether both web reinforcement yield or diagonal concrete strut crush

```

If (zheta * fc) >= (-sigmav - sigmah + sigmar + pwv *
Application.WorksheetFunction.Min(0.8 * fywv, 500) + pwh *
Application.WorksheetFunction.Min(0.8 * fywh, 500)) Then
    sigmad = -1 * (-sigmav - sigmah + sigmar + pwv *
Application.WorksheetFunction.Min(0.8 * fywv, 500) + pwh *
Application.WorksheetFunction.Min(0.8 * fywh, 500))
    fwv = Application.WorksheetFunction.Min(0.8 * fywv, 500)
    fwh = Application.WorksheetFunction.Min(0.8 * fywh, 500)
Else
    sigmad = -zheta * fc
    If (Hw / Lw) < 1 Then
        fwv = Application.WorksheetFunction.Min(0.8 * fywv, 500)
        fwh = (-sigmad + sigmav + sigmah - sigmar - pwv * fwv) / pwh
    Else
        fwh = Application.WorksheetFunction.Min(0.8 * fywh, 500)
        fwv = (-sigmad + sigmav + sigmah - sigmar - pwh * fwh) / pwv
    End If
End If

'Defining theta
theta = Application.WorksheetFunction.Asin((( -sigmav + sigmar + pwv * fwv)
/ (-sigmad + sigmar)) ^ 0.5)

'Defining dowel action
If bf <= 0.5 * Hw + tw Then
    bfuse = bf
Else
    bfuse = 0.5 * Hw + tw
End If
Asb = pb * bfuse * tf
dbe = (Asb / (0.25 * Application.WorksheetFunction.Pi())) ^ 0.5
Du = 1.64 * (bfuse - dbe) * dbe * (fc ^ (1 / 3)) / 1000

```

$V_{\text{proposedmodel}} = (\sigma_{\text{mar}} - \sigma_{\text{mad}}) * \sin(\theta) * \cos(\theta) * t_w * d_w / 1000 + D_u$

If behavior = "SHEAR" Then

Worksheets("All").Cells(i, "QK").Value = $V_{\text{proposedmodel}}$

Worksheets("All").Cells(i, "QL").Value = $V_{\text{exp}} / V_{\text{proposedmodel}}$

End If

Next i

,

End Sub

C.7 Proposed Strut-Tie Model

Sub StrutandTieModel()

,

' Strut and Tie Model for Shear Strength of RC Wall

,

Dim i As Integer

For i = 3 To 86

'Defining input variables

f_c = Worksheets("All").Cells(i, "D").Value 'concrete cylinder compressive strength (in MPa)

A_g = Worksheets("All").Cells(i, "E").Value 'gross area of wall cross section (in mm²)

P = Worksheets("All").Cells(i, "F").Value 'axial load applied (in kN)

H_w = Worksheets("All").Cells(i, "J").Value 'clear height of wall (in mm)

L_w = Worksheets("All").Cells(i, "K").Value 'length of wall (in mm)

d_w = Worksheets("All").Cells(i, "L").Value 'effective depth of wall (in mm)


```

tw = Worksheets("All").Cells(i, "M").Value 'thickness of wall web (in mm)
bf = Worksheets("All").Cells(i, "N").Value 'boundary element width (in mm)
tf = Worksheets("All").Cells(i, "O").Value 'boundary element thickness (in
mm)
Hprime = Worksheets("All").Cells(i, "P").Value 'distance of point of
application of shear force to wall base (in mm)
pb = Worksheets("All").Cells(i, "Q").Value 'boundary element vertical
reinforcement ratio
fyb = Worksheets("All").Cells(i, "R").Value 'boundary element vertical
reinforcement yield strength (in MPa)
pwv = Worksheets("All").Cells(i, "W").Value 'wall web vertical reinforcement
ratio
fywv = Worksheets("All").Cells(i, "X").Value 'wall web vertical reinforcement
yield strength (in MPa)
pwh = Worksheets("All").Cells(i, "Y").Value 'wall web horizontal
reinforcement ratio
fywh = Worksheets("All").Cells(i, "Z").Value 'wall web horizontal
reinforcement yield strength (in MPa)
Vexp = Worksheets("All").Cells(i, "AA").Value 'experimental wall strength
(in kN)
drift = Worksheets("All").Cells(i, "AB").Value 'drift ratio at experimental wall
strength
behavior = Worksheets("All").Cells(i, "AC").Value 'failure mode

'Calculating c (depth of compression zone)
c = Application.WorksheetFunction.Min(Lw * (0.35 + 0.5 * P * 1000 / (fc *
Lw * tw) + 6 * pb * bf * tf / (Lw * tw)) * (Hw / Lw) ^ (-0.4), dw)

'Calculating r (distance from resultant force R = P+T to the nearest wall edge)
T1 = pb * bf * tf * fyb
r1 = 0.5 * tf
If (Lw - c - tf) > 0 Then

```

```

    T2 = pwv * (Lw - c - tf) * tw * fywv
    r2 = tf + 0.5 * (Lw - c - tf)
Else
    T2 = 0
    r2 = 0
End If
r = (T1 * r1 + T2 * r2 + P * 1000 * 0.5 * Lw) / (T1 + T2 + P * 1000)

'Calculating strut angle
theta = Atn(Hprime / (Lw - r - c / 2))
theta_min = 0.541052 'setting minimum theta, not less than about 31 degrees
theta_max = 1.029744 'setting maximum theta, not more than about 59 degrees
If theta < theta_min Then
    theta = theta_min
End If
If theta > theta_max Then
    theta = theta_max
End If

'Calculating zheta (softening factor for diagonal compression strut)
alpha_w = 0.4
pfy = pwv * fywv * Cos(theta) + pwh * fywh * Sin(theta)
omega_w = 4 * pfy / fc
zheta = Application.WorksheetFunction.Min(0.85, 0.6 * (1 - fc / 250) * 0.8 *
(1 + 1.6 * alpha_w * omega_w))

'Calculating Astr (area of diagonal compression strut)
Astrut = tw * c * Sin(theta)

'Calculating Dstrut (capacity of diagonal compression strut)
Dstrut = zheta * fc * Astrut

```

'Calculating Vstrut (shear strength due to strut failure)

$V_{strut} = D_{strut} * \cos(\theta) / 1000$

'Ratio with experimental results (specimens failing in shear)

$V_{exVstt} = V_{exp} / V_{strut}$

'Sending shear strength value and ratio to excel worksheet

Worksheets("All").Cells(i, "QM").Value = Vstrut 'shear failure load'

Worksheets("All").Cells(i, "QN").Value = VexVstt 'ratio of shear failure loads'

Next i

End Sub