Design of reinforced concrete walls with openings for strength and ductility

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Design of Reinforced Concrete Walls with Openings for Strength and Ductility

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School of Civil & Environmental Engineering

**A thesis submitted to the Nanyang Technological University
in fulfilment of the requirements for the degree of
Doctor of Philosophy**

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ABSTRACT

To date limited research on the seismic behaviour of walls with irregular openings has been conducted, and a general design recommendation based on the strut-and-tie models and the capacity design procedures has been provided by the ACI 318 [AI] and NZS 3101 [N6] codes. However, the behaviour of the structural walls with irregular openings subjected to seismic loadings and the details of the design procedures have not been adequately addressed. Therefore, experimental and analytical studies on the seismic design of walls with irregular openings were performed in this study.

Six flanged walls, consisting of one solid wall, three walls with irregular openings, and two walls with regular openings were tested under reversed cyclic loadings. It was found that the flanged walls with irregular openings could perform similar to the solid flanged wall. All of the walls reached the flexural yield strength before failures. The modified strut-and-tie models developed based on the load paths of the walls with irregular openings could predict the behaviour of the walls satisfactorily; however, because of the presence of flanges, the walls often failed in sliding failure modes.

A series of parametric studies on the walls with openings were performed using a reliable non-linear finite element programme. The selected parameters included the flanges, the axial loadings, the different loading schemes, the reinforcements in the different zones of the walls with irregular openings, and the size and arrangement of the openings etc. Both the quasi-static analysis and the dynamic analysis were carried out.

In addition, a seismic performance evaluation on walls with openings was performed using the capacity spectrum method. The seismic demands of these walls were investigated.

Based on the experimental and analytical results, a general design approach was proposed for the design of walls with irregular openings. In this design method, flexural behaviour of the walls was studied for the selections of strut-and-tie models and the determinations of overstrengths for the shear design, so that the undesirable failure modes could be avoided.

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LIST OF SYMBOLS

a	Axial loading ratio
A_c	Net section area
A_g	Gross area of wall section
A_h	Area enclosed by the hysteretic loop
A_{RC}	Area of RC zone surrounding the reinforcement
A_s	Area of reinforcement of a tie
A_{st}	Area of reinforcement effectively bonded to concrete in tension
A_v	Area of shear reinforcement within a distance s
b_w	Web width of wall section
C	Stiffening/softening factor
C_A, C_V	Seismic coefficients
D	Roof displacement
d_b	Diameter of the reinforcing bar
DF	Displacement ductility factor used in the test
d_{max}	Maximum aggregate size
E_c	Concrete elastic modulus,
E_s	Young's modulus of steel bar
E_{sh}	Hardening modulus of steel bar
f'_c	Concrete cylinder compressive strength
f'_{ce}	Effective strength of the concrete struts
f'_{cu}	Concrete cube compressive strength
F_m	Peak loading of the loop
f_t	Concrete tensile strength
f_y	Yield strength of steel bar
f_y^*	Average apparent yield stress of stiffened reinforcing bars
f_u	Ultimate strength of steel bar

G	Shear modulus of concrete
G_c	Concrete compressive fracture energy
G^{cr}	Concrete shear modulus after cracking
G_f	Concrete fracture energy
G_{f0}	Coefficients for determination of the fracture energy
h	Crack band width of the element
h_i	Vertical distance between the centre line of region i to the wall base
H_i	Vertical distance from the centre line to the loading point
$h_{\max}$	Maximum height of the RC zone
h_w	Height of wall
I_e	Equivalent moment of inertia
I_g	Gross moment of inertia
K_1	Stiffness of a wall at $DF = 1$
K_i	Initial stiffness
L	Length of the diagonal
l_{c2}	Distance between the compression force C_1 and C_2
L_T	Horizontal distance between left and right LVDTs
L_w, l_w	Horizontal length of wall
l_h	Clear span of coupling beam
M^*	Design moment at the section at eth ultimate limit state
M_{Bo}	Flexural overstrengths of wall
m_i	Mass lumped on level i
M_i	Nominal flexural strength
M_{pl}	Moment calculated according to plane section hypothesis
M_u	Moment capacity of the section
n	Level n , the uppermost level of the structure
N^*	Design axial load at the section at eth ultimate limit state

P	Tensile yield force of a tie considering concrete tensile strength and tension stiffening on reinforcing bar
	Modal participation factor for the first natural mode
	Lateral force
P_e	Experimental maximum lateral loading
P_f	Maximum lateral loading computed
P_i	Ideal strength of specimen
$P_{j\text{design}}$	Design lateral force of j th specimen
$P_{j\text{max}}$	Maximum recorded lateral force of j th specimen
P_y	Top lateral loading corresponding to first yield of one of tie
P_u	Axial compression force
R	Reduction factor due to ductility for inelastic spectrum
s	Spacing of horizontal reinforcement in wall
S_a	Spectral acceleration
S_{ae}	Elastic spectral acceleration (g)
$S_{a\mu}$	Spectral acceleration of intersection. Its value should be constant after yield point because the capacity diagram is in the bilinear format.
S_d	Spectral displacement
S_{de}	Elastic spectral displacement (m)
S_{dy}	Yield spectral displacement in capacity diagram
$S_{d\mu}$	Spectral displacement of the intersection.
S_i	Gauge length of LVDT in region i
S_n	Nominal strength
S_u	Design strength
T	Period (s)
T_μ	Corresponding period
V	Volume of the element
	Shear force
V^*	Design shear force at the section at eth ultimate limit state

V_c	Concrete contribution for the shear capacity of a column
V_{cr}	Strengths at the onset of diagonal cracks
V_E	Experiment maximum shear forces
V_M	Maximum allowable shear forces controlled by the moment capacity of columns
V_P	Shear capacities of piers calculated
V_s	Reinforcement contribution for the shear capacity of a pier.
w_s	Effective strut width
w_t	Effective tie width
α	Angle between the diagonal and the side of rectangle Loading factor of the tie in the strut-and-tie model Strain hardening ratios
α_1	Angle shown in Figure 3.5.17 Modal mass coefficient for the first natural mode
α_2	Angle shown in Figure 3.5.17
β	Shear retention factor
δ_{Li}	Extension and shortening of left flange measured by a pair of LVDTs in the region i
$\delta_{\max}$	Maximum top drift ratio achieved
δ_{Ri}	Extension and shortening of right flange measured by a pair of LVDTs in the region i
ε	Average tensile strain
ε_1	Principal tensile strain at right angles to the direction of the strut
ε_2	Principal compressive strain in the direction of the strut
ε_s	Yield strain of the reinforcement
ε_y	Yield strain of reinforcement
ε_r	Average principal tensile strain in the tensile principal axis (r) direction
γ	Shear distortion of wall
θ	Angle of the resultant compressive strut
θ_1	Angle shown in Figure 3.5.17

List of symbols

θ_2	Angle shown in Figure 3.5.17
θ_i	Change of slope caused by flexure deformation of wall occurring in the region i
λ	Factor which depends on the concrete density
λ_1	Maximum proportion of shear force carried by the section 1-1
λ_2	Maximum proportion of shear force carried by the section 2-2
λ_o	Overstrength factor for steel
λ_f	Flexural reduction factor due to the shear transfer
λ_p	Shear proportion taken by bottom panel
μ	Ductility factor
μ_Δ	Displacement ductility factor
ν_c	Concrete Poisson ratio
ρ	Ratio of steel area to the net concrete area
ρ_{eff}	Effective reinforcement ratio
ρ_v	Vertical reinforcement ratio
ρ_w	Ratio of the flexural tension reinforcement
σ	Average tensile stress
σ_1	Transverse compressive stress
$\sigma_{c\ max}$	Strength developed in the strut when maximum strength of model is reached
ϕ	Strength reduction factor
ϕ_c	Material factor for concrete
$\phi_{o,w}$	Flexural overstrength factor
ϕ_{i1}	Amplitude of mode 1 at level i
ξ	Effective factor which is less than unit
ξ_{hyst}	Hysteretic damping
ξ_r	Tensile strain
Δ	Top drift imposed to the structure
Δ_1, Δ_2	Deformation of the panel diagonals
$\Delta_{0.75}$	Top drifts obtained from $\pm 0.75P_i$

Δ_f	Deformations make contributions to the top drift of wall caused by the flexure Top drift induced by shear distortion
Δ_{fi}	Top drift induced by shear distortion in region i
Δ_R	Deformations make contributions to the top drift of wall caused by the rotation
Δ_s	Deformations make contributions to the top drift of wall caused by the sliding
Δ_v	Deformations make contributions to the top drift of wall caused by the shear
Δ_y	Structural yield displacement
δ	Drift ratio

CHAPTER ONE

INTRODUCTION

1.1 Problem Definition

Structural walls can provide structures under service loading with sufficient stiffness, minimizing deformations and damage to non-structural elements. Under severe seismic excitations, they can provide sufficient strength, energy absorption and dissipation capacities to prevent collapse and loss of life. In order to fulfill the need of accommodating for windows, doors or utility ducts, walls are often pierced with openings. If the openings are small relative to the wall dimensions, it is usually reasonable to ignore the effects of the openings. However, in the cases that the openings are relatively large or are located in a critical region, the openings may significantly impose influences on the behaviour of a reinforced concrete wall, such as changing its force transfer mechanism, deducting its strength and stiffness, and decreasing its ductility level.

Most of the research on walls has been focused on the design of walls without openings or with symmetric openings, which are often regarded as coupled wall system. Only a little research has been done, however, on the walls with irregular openings by researchers, such as Ali [A6] and Yanez *et al* [Y1], after this kind of walls have shown satisfactory performance in severe seismic regions [W10]. Their research has also indicated that this kind of walls could achieve similar behaviour to the solid wall, provided the walls were well-detailed. Based on these, the ACI 318 [A1] and NZS3101 [N6] design codes allow the utilisation of the walls with irregular openings, but a precondition that they are appropriately analyzed and detailed is required. The strut-and-tie models and the capacity design procedures, proved by previous researcher as effective methods on the designed this kind of walls, are suggested as a basis for their designs. However, their research at the same time has indicated that the strut-and-tie model may severely underestimate the strength of the walls, and detailed guidelines for using capacity procedure on the design of this kind of wall have not been given. Therefore, more experimental and theoretical research programmes on the structural walls with irregular openings are still needed, to investigate their strength, stiffness and failure mechanisms *etc*, so that their design methodology can be developed and detailed.

Walls with irregular openings have discontinuous regions according to the information gathered from stress analyses. Hence, the strut-and-tie models appear to be appropriate for their analyses and designs. The strut-and-tie models first proposed by Ritter [R3] and Morsch [M11] about a century ago were the generalisations of the truss analogy, which have been refined and expanded since the 1960s [A2, S2]. Currently, this methodology has become a powerful tool in the investigation of the behaviour of reinforced concrete structures, especially in regarding the design of reinforcement in the discontinuous regions of the structures. Some attempts to design walls with irregular openings using the strut-and-tie models have been made and satisfactory results have been obtained [Y1, T1].

Current research on the seismic design of reinforced concrete structures tends towards a performance-based design, in which the seismic loading is related directly to the performance of the structures. Some performance-based methods, such as the capacity design method [P3, P7] and displacement-based design method [W1~W3], have been proposed and proved to be more efficient on the seismic design of reinforced concrete structural walls than the force-based design method. However, most of the strut-and-tie models were developed for non-seismic loading cases and only for the force-based design. Only few attempts to investigate the stiffness, ductility and hysteretic response of reinforced concrete structures using the strut-and-tie models have been conducted recently [K4, T3]. Consequently, whether the application of the strut-and-tie models on the seismic design of reinforced concrete structures can achieve the aims of the performance-based design or not still need to be proved. This is also one of the objectives of this research. Therefore some attempts using strut-and-tie model to predict the stiffness of the walls are also made.

As a whole, in this study an experimental programme on the walls with irregular openings has been performed. The predicted behaviour of the walls based on the strut-and-tie models will be compared with their experimental behaviour. Based on the experimental results, the design method incorporating the strut-and-tie models and the capacity design procedure will be examined, detailed and modified.

1.2 Objectives and Scopes of Research

The objective of this study is to investigate the seismic behaviour of reinforced concrete walls with irregular openings upon which design recommendations can be based. The

scope of this study consists of the following tasks. The first task involves an experimental programme on the walls with irregular openings; therefore, six flanged walls, including one solid wall, three walls with irregular openings and two walls with regular openings will be tested under reversed cyclic loadings. The second task involves the nonlinear analyses of the reinforced concrete structural walls using the Finite Element Method (FEM). A series of parametric studies, including a quasi-static analysis and a dynamic analysis, on the walls with openings using reliable nonlinear finite element analysis programmes will be performed to observe the effects of the selected parameters on the strength, stiffness and ductility of these walls. The parameters, including the flange, the axial loading, the loading schemes, and the size and arrangement of the openings, will be investigated. The third task includes the development of strut-and-tie models for the walls and the comparison between the behaviour predicted by the models and the experimental ones. Finally a seismic performance evaluation on the walls with openings according to the experimental results will be performed and a design method based on the strut-and-tie models and the capacity design procedures will be proposed.

1.3 Organization of Thesis

This thesis is divided into eleven chapters. Chapter 1 provides an introduction to the background, the objectives and the scope of this research. Chapter 2 presents a literature review of the recent research involving reinforced concrete structural walls with irregular openings and the strut-and-tie models. The capacity design philosophy is also reviewed in this chapter. Chapter 3 introduces the modeling methods of reinforced concrete walls using nonlinear finite element analysis programmes and the verification of the models using existing test results. In Chapter 4, the details of the experimental programme are presented. The experimental results are reported thoroughly in Chapter 5 and are discussed in Chapter 6. The static parametric study is performed in Chapter 7, followed by the dynamic analysis conducted in Chapter 8, and a seismic performance evaluation on the walls with openings is carried out in Chapter 9. Finally, the design recommendations are suggested in Chapter 10 and the conclusions are reached in Chapter 11, in which some recommendations worthy of future research are also included.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

To augment the work done in this study, a review of previous experimental and analytical investigations on structural walls with openings was required. The scope of this chapter, therefore, deals with a literature review of the related work done on the walls. The strut-and-tie model and the capacity design procedures, suggested by ACI 318 [A1] and NZS 3101 [N6] as the design guidelines for the walls with irregular openings, are also summarised. This investigation will provide information on the behaviour of this kind of walls under reversed cyclic lateral loading and the backgrounds of their design methods.

2.2 Previous Research on the Walls with Irregular Openings

2.2.1 Research Conducted by Ali and Wight

Research conducted by Ali and Wight [A4~A6] focused on determining the effects of different staggered configurations for door openings on the strengths, stiffness, and deformation capacities of structural walls under reversed cyclic loads. Four five-story walls with staggered openings were constructed at approximately one-fifth scale. The details of the walls are presented in **Figures A.1 to A.5**. The walls had barbell shaped cross sections and were lightly reinforced. Three of the walls had an opening at each storey level with varying amounts of horizontal stagger between the openings, which was the only variable involved in the study. A wall without openings was tested for comparison. The walls were modeled loosely upon typical Chilean construction, which was found to perform well during a severe earthquake there in 1985 [W10]. Each wall was 3.56 m high and 1.22 m wide, resulting in an aspect ratio of 2.9. The boundary elements were 127×127 mm reinforced with 4-#4 (12 mm) bars. The width of the web was 76 mm with a single curtain of #2 (6 mm) bars at 152 mm. The openings were 229×508 mm making up 13.4% of the gross wall area. The reinforcement of the tested walls was selected primarily based upon the code extrapolation and the design judgment. Hoops were provided at 63.5 mm in the boundary elements using 4.8 mm diameter wire, resulting in a transverse reinforcing ratio of 0.004. Short diagonal 4.8 mm diameter bars

were placed at the corners of the openings to control cracking.

The walls were tested under reversed cyclic loading applied at their top while a constant axial load of approximately $0.07A_g f'_c$ was applied. Ductile flexural behavior was observed up to a drift of 1.0%. At a drift between 1.25% and 1.5%, all specimens with openings experienced a sudden shear compression failure in the “column” next to the openings at the first storey level, which was characterized by crushing of the concrete in the boundary element and buckling of the main longitudinal reinforcement. No shear distress was observed between the staggered openings.

The following conclusions were drawn for lightly reinforced walls with a moderate level of axial load and shear stress.

- 1) 1.0% lateral drift may be obtained without significant damage and higher drifts may be obtained with greater confinement.
- 2) The Uniform Building Code (1985) [I1] requirements for boundary element confinement can be significantly relaxed for these walls.
- 3) Staggered openings are a viable alternative to in-line openings.
- 4) Door openings located too close to the boundary element can trigger a shear compression failure.

2.2.2 Research Conducted by Yanez, Park and Paulay

Research conducted by Yanez *et al* [Y1, Y2] focused on the seismic behavior of reinforced concrete walls with irregularly distributed openings. A design procedure using strut-and-tie models was proposed and verified. Six three-storey reinforced concrete model walls, scaled to one third, were tested under reversed cyclic lateral loading. Three of the walls had irregular openings, two had regular openings, and one had no openings. The details of these six walls are displayed in **Figures A.6 to A.10**. The vertical and horizontal reinforcing ratios for all walls were approximately 0.5% and 0.4% respectively. The walls were 2 m wide, 2.3 m high and 120 mm thick, resulting in an aspect ratio of 1.25. Reversed cyclic loading was applied at the top of the walls with no axial load being applied.

The strut-and-tie models were used to design the three specimens having irregular openings. It was found that although the results of these tests showed that the strut-and-tie models significantly underestimated the lateral load capacities of the walls, the

superposition of the two strut-and-tie mechanisms (one for each direction of loading) was still valid because all specimens could reach a displacement ductility level between 5 to 6 and the strut-and-tie models used for the design of the walls displayed a good relationship with the crack patterns of the walls. The main vertical tension bars fracturing and the compressed concrete crushing at a foundation level were observed when the walls failed.

It was concluded that the arrangement of the openings did not have a significant effect on the seismic behavior of the walls, although the size of the openings was important, and the use of a strut-and-tie model could be an effective tool in the design of a low aspect shear wall, in which behavior is dominated by shear.

2.2.3 Research Conducted by Taylor, Cote and Wallace

Research conducted by Taylor *et al* [T1] focused on the design of slender reinforced concrete structural walls with an opening at the base, where inelastic deformations were concentrated. The design procedure involved a displacement-based design approach and a strut-and-tie concept. The design was broken down into a series of steps in which each type of reinforcement was selected. First, a boundary longitudinal reinforcement was selected based on code prescribed forces and on a standard cross sectional analysis, which assumed that plane sections remain plane. Second, the transverse reinforcement was selected using the displacement-based design procedures. Finally, the horizontal shear reinforcement was selected using capacity design procedure and strut-and-tie models.

An experimental study was performed to verify that this design procedure was appropriate for evaluating and detailing the shear requirements of slender walls with a relatively large opening at the base of the wall. Two four-storey specimens were constructed at one quarter in scale, one with a rectangular cross section and the other with a barbell shaped cross section, whose details are shown in **Figures B.1 to B.4** of Appendix B. Materials used were 27.6 MPa concrete and GR 60 (414 MPa) steel. The walls were 3.66 m high and 1.22 m wide, resulting in an aspect ratio of 3. Each wall contained confined boundary regions. Horizontal reinforcing and vertical reinforcing of each wall varied with the height of the wall due to the interruption of shear flows within the web, which was caused by the openings. A reversed cyclic loading was applied slowly to the top of the specimens after an axial load of approximately $0.10A_g f'_c$ was applied to the wall, which were held constant for the duration of the whole test.

Experimental results indicated that both of the walls displayed adequate ductility at up to

1.5 percent lateral drift. The main longitudinal bars of the two specimens yielded at 0.75 percent lateral drift and some of them buckled at the failure stage. The actual stress in the longitudinal steel was about 75 to 100 percent of the stress predicted by the strut-and-tie models used in the design. It was concluded that the properly reinforced slender wall with an opening at the base could exhibit stable hysteretic behavior and significant ductility even in the case where the openings were in the flexural compression zone. The large openings at the base did not significantly influence the behavior of the wall compared with the results of walls without openings. The strut-and-tie models and the displacement-based design approach were effective for assessing and detailing the walls with an opening in the base.

2.2.4 Research Conducted by Kato and Noda

A series of studies on the walls with an opening have been finished by Kato *et al* [K2] in the last two decades. The aims of these studies focused on applying an ultimate strength concept on the design of shear walls with openings. Based on these studies, a rational design method was proposed. The following conclusions were reached.

- (1) The location of the failure zone of a wall with an opening was concentrated compared with that of a wall without an opening. Generally, sidewalls on both sides of the opening were apt to fail.
- (2) If enough shear reinforcement was provided for both sidewalls, the wall failed in flexure and showed ductile behavior similar to walls without openings.
- (3) Diagonal reinforcing bars in the sidewalls were effective in enhancing the shear strength of the wall with an opening.
- (4) The strength of the wall with an opening was a summation of the strengths of both sidewalls, which took into account the contribution of the peripheral reinforcement around the opening.

The other series of studies on the shear walls with multiple openings were completed by Noda *et al* [N5] and Kato *et al* [K3]. The main objectives of the studies were to examine the effects of wall openings on the strength and deformation capacities of the cantilever shear walls. Six reinforced concrete cantilever shear wall specimens with openings, which were designed to fail in flexure, were tested under static reversed loading. Variables in the six specimens were the peripheral ratios of the panel (0.3, 0.4 and 0.5), the sizes and locations of the openings, and the reinforcement of the sidewalls next to the openings. All

specimens were two-storey single-bay walls, but the top girders were intentionally made very stiff and strong in order to apply uniform loads to the wall panels as well as to simulate the confining effect of multi-storey shear walls. **Figure B.5** in Appendix B displays the reinforcement details of four typical specimens. Three types of specimens including the central window opening type, the central door opening type and the eccentric door opening type were tested. An axial load of 150 kN was added during the test. The reversed cyclic lateral loading was applied at the end of the top girder. Each specimen showed ductile behaviour up to a top drift angle of 1%, and the pinching-type behaviour was observed in the hysteretic loop for further loading exceeding 1%.

Strut-and-tie models were used to investigate the effects of the openings on the behaviour of the structural walls. Simplified strut-and-tie models for walls with an opening were proposed. Based on the shear force transfer mechanisms derived from the strut-and-tie models, shear walls were classified into two types: the F wall, meaning the flexural type, and the FS wall, referred to the flexural shear type. Different design methods were proposed for each type of wall. If the wall was an F type wall, the deformation capacity could be obtained through the effectiveness of the concrete with the proposed design equation for shear using a sidewall in compression only. If the wall is a FS type wall, the deformation capacity could be obtained through the effectiveness of the concrete using the summation of both sidewalls. Thus this deformation capacity should be reduced by the effects of the openings.

2.3 Strut-and-Tie Models

2.3.1 Development and application

Strut-and-tie models, being the generalization of truss models, demonstrate the rational characteristics of the mechanical properties of reinforced concrete structures, which involve utilizing concrete struts to resist compression and reinforcing ties to carry tension, and designers have used this concept intuitively for many years. At the turn of the twentieth century, Ritter [R3] and Morsch [M12] introduced the concept of truss analogy in the shear design of reinforced concrete beams known as “45°-angle truss model”, which is still the basic model for shear design [A1]. Its schematic diagram is presented in **Figure 2.3.1**. The truss analogy concept was extended in the late 1920s by Rausch [R2] as a means of designing reinforced concrete beams to resist torsion. Since then, no

significant contributions had been made to the development of strut-and-tie models until Kupfer [K6] in the 1960s, found an equation for the inclination of the diagonal cracks by minimizing the strain energy of the whole truss model in the 1960s. Kupfer's work contributed to the design method adopted in the European Code CEB-FIP (1978) [C4]. Using the same assumptions, Baumann [B1] derived a similar equation for the crack direction in reinforced concrete panels, which had been subjected to plane stress states. At the same time, Leonhardt [L3] proposed a truss model with a variable angle of inclination for the compression struts. This concept was often applied to simulate the fan-shaped stress field by later researchers [M5, P4, P5]. **Figure 2.3.2** schematically plots the variable angle truss model utilized by Paulay [P5] to define the contribution of truss action in the thin webbed coupling beam.

Figure 2.3.1 45° angle truss model

Figure 2.3.2 Variable angle truss model

The truss model was further modified by using the theory of plasticity. Nielson [N2~N6] and Lampert and Thurlimann [L1] initiated the use of membrane elements in the analysis of reinforced concrete structures. The application of limit analysis methods capable of predicting failure-conditions in the truss model approach started in 1970s. Elfgren [E1] developed a truss model that was called the “compression stress field theory” by assuming that both the longitudinal and the transverse steel yield at the ultimate load stage. Using a variable angle space truss approach, Lampert and Thurlimann [L2] studied beams subjected to torsion and flexure. Thurlimann *et al* [T2] extended the work to cover warping torsion as well as combined flexure, shear and torsion. Thurlimann's space truss model became the basis of the shear and torsion design method adopted in the 1978 CEB-FIP design code [C4].

Before 1972, the truss model severely overestimated the shear and torsion strength because the failure stress of the concrete struts was assumed to be the uniaxial compression strength of standard concrete cylinders. This situation began changing after Robinson and Demorieux [R4, R5] observed that a reinforced concrete element under shear is actually subjected to a two-dimensional stress condition. The strength from the principal compression direction was found to be softened by the principal tension in the perpendicular direction. This softened stress was referred to as the “effective stress of concrete”. Vecchio and Collins [V1] proposed the first softened stress-strain curve of concrete in compression to determine the “effective stress”. Combining the softened stress-strain curve of concrete with Mohr’s stress and strain circles, they developed a “compression field theory” to predict the behavior of reinforced concrete elements. This theory was modified in an attempt to derive the “contribution of concrete” [V2], which is called “modified compression field theory (MCFT)”. In order to improve the simulation of responses in specific situations where the MCFT was found to produce inaccuracies, a disturbed stress field model (DSFM) was proposed by Vecchio [V3] in 2000, in an attempt to redress the two main weaknesses in the MCFT computational model: the enforced alignment of the principal stress and strain directions and the handling of crack shear stresses. Based on the equilibrium, compatibility and the constitutive laws of material, a truss model called a “softened truss model” was also developed by Hsu [H5, H6] and Pang and Hsu [P1], and a unified theory was proposed for the behavior of reinforced concrete structures subjected to bending, axial load and torsion [H6]. In 1998, Kaufmann and Marti [K1] proposed a new model named the “cracked membrane model” for cracked, orthogonal reinforced concrete panels, which have been subjected to in-plane stresses, and which combines the basic concepts of the modified compression field approach and the tension chord model [M9].

Marti [M4~M8, M10] and Schlaich *et al* [S2, S3] further clarified and greatly promoted the applications of the strut-and-tie models to the design and detailing of reinforced concrete structures. Marti [M5] provided a comprehensive synthesis of the truss model approaches and related theories used in the detailing and dimensions of the required structural concrete members. A set of basic tools, which include struts and ties, nodes, fans, and arches, is described in his work. He emphasized that the strut-and-tie models and the stress field considerations should be part of the toolbox of every designer. Schlaich *et al* [S2, S3] extended the truss model approach to overall structures in the form

of strut-and-tie systems by defining the B-and D-regions in any reinforced or prestressed concrete structure. A series of methods for the development of the strut-and-tie systems were proposed.

In the last two decades, the strut-and-tie models have been successfully applied to design and detail the many types of non-flexural reinforced concrete structures, including deep beams, beam supports, knee joints, corbels, anchorage regions and membrane elements with openings. Some typical applications are briefly reviewed in this section.

Rogowsky and MacGregor [R6] presented a design method of deep beams using the strut-and-tie model. The predicted behaviour gives good agreement with the test results of deep beams. They suggested that the strut angles should be limited to between $25^\circ \sim 65^\circ$ and that the major struts should be within $\pm 15^\circ$ of the elastic compressive stress trajectories.

Paulay [P6] applied the truss analogy in seismic design of building joints. He found that joint shear reinforcement is necessary to sustain a diagonal compression field rather than to provide confinement to compressed concrete in a joint core. His finding was adopted in the NZS3101 code [N6].

When strut-and-tie models were used to improve the design of pile caps, Adebar *et al* [A3] found that they could more accurately describe the deep pile caps behaviour than had the ACI 318 [A1] procedures. The influences of the amount and the horizontal distribution of longitudinal reinforcement were emphasized according to their research.

Hong *et al* [H2~H4] developed the truss models for the design of the negative moment regions and the CCT and TTC nodes (referred to Section 2.3.5), which are based on the classical theory of plasticity, in particular the normality rule, and the lower and upper bound theorems. The models identified different failure modes related to bond strength and dependence on material strength and geometry.

Xiao, Priestley and Seible [X1] used strut-and-tie model and yield-line theories for the seismic assessment and retrofit of bridge column footings. In their study, different strut-and-tie models were developed for investigating force transfer in the pre-cracking and post-cracking footing respectively. This model concept was also adopted for seismic design and assessment of concrete bridge joints by Sritharan *et al* [S5].

Ramirez [R1] presented guidelines for the strut-and-tie design of pretensioned concrete members, using a full member strut-and-tie approach.

Hwang *et al* [H8, H9] proposed the softened strut-and-tie models to predict the shear strength of the exterior and interior reinforced concrete beam-column joints for seismic resistance. The concept of this model was also used to predict the shear strength of reinforced concrete deep beams and squat walls [H10, H11]. Every predicted result showed a reasonable agreement with the corresponding experimental result.

Kim and Mander [K4, K5] did extensive investigation into two truss models: a constant crack angle truss and a variable angle truss. The constant angle truss was considered appropriate for an undisturbed region of sufficiently long elements, while the variable angle truss was for squat elements and the disturbed end-regions of long elements. A strut-and-tie model was developed to take a close look into the post-elastic deformation capacity of a beam-column connection in ductile reinforced concrete frame structures, which was regarded as a disturbed region [K5]. The effect of concrete strength decay was also considered in their study.

Some attempts to develop strut-and-tie models that allow for the cyclic hysteretic response of the reinforced concrete structure have been made. In 1996, Kim and Mander [K4] developed a CIST (cyclic inelastic strut-and-tie) model for the analysis of Paulay's [P4, P5] coupling beam "Specimen 312". Satisfactory agreement between the theory and the experiment was shown though the concrete strength decay was not considered in the analysis. A cyclic strut-and-tie model was developed by To *et al* [T3] in 2000 to investigate the reinforced beam under reversed cyclic loading. The result was found to satisfactorily match the experimentally measured force-displacement response.

Developing strut-and-tie models for the structural design with the aid of a computer is a trend in the development of this method. Schlaich *et al* [S4] developed an algorithm for the automatic generation of "transition stress fields", which was a concept concerning the stress fields of nodes in strut-and-tie models. Alshegeir, Yun and Ramirez [A8, Y3, Y4] developed interactive computer graphics programs to implement the strut-and-tie model approach for the analysis and design of reinforced and prestressed concrete members including the D-regions. A method for automatic generation of truss model for optimal design reinforced concrete structures was also proposed by Ali *et al* [A7].

On the whole, the strut-and-tie models so far developed have become a powerful tool in the design of reinforced concrete structures and the development of this method is still going on. This method has been incorporated into Appendix A of the ACI 318 design code [A1].

2.3.2 B-regions and D-regions

Those regions of a structure in which Bernoulli's hypothesis of plane distribution of strain can be applied have been referred to as B-regions by Schlaich [S2, S3]. In these regions if the concrete remains uncracked, the stress is calculated using the bending theory for linear elastic material. For the cracked B-regions the truss models or the standard methods of codes can be applied.

Those regions where the strain distribution is significantly nonlinear, for example near concentrated loads, corners, bends, openings and other discontinuities, are called D-regions. In these regions, the standard methods are not applicable.

An advantage of dividing a structure into B- and D-regions is that the attention of the designer is focused on the potentially weak spots that require special attention in detailing and design. However, not much accuracy is necessary in determining the dividing sections between B- and D- regions. These sections can be assumed to lie approximately in a distance h from the geometrical discontinuity or the concentrated load, where h is equal to the depth of the adjacent B-region. This assumption is justified by Saint-Venant's principle.

2.3.3 Stress fields and load path method

Since the struts in the strut-and-tie models are resultants of the stress fields, it is very important to know the properties of the compression stress fields in the reinforced concrete structures. In 1987 Schlaich *et al* [S2] provided three typical configurations to cover all cases of compression stress fields:

- (1) The fan-shaped (**Fig 2.3.3 a**) stress is an idealization of a stress field with negligible curvature. This field can occur where a force is introduced and channelled through an element that is itself fan-shaped. There is no transverse tensile stress field existing in the fans.
- (2) The bottle-shaped stress field (**Fig 2.3.3 b**), with its bulging stress trajectories, develops considerable transverse stress. A bottle-shaped stress field occurs wherever a distributed stress field is equilibrated by a narrower, more concentrated stress field. In any bottle-shaped field there are associated transverse tensile stresses, which have to be carried, either by the concrete in tension, or by specially introduced transverse reinforcement.
- (3) The prismatic or parallel stress field (**Fig 2.3.3 c**) is the simplest compressive

stress field. Without any narrowing or splaying of the stress trajectories, there are no associated transverse tensile stresses induced in the concrete. The compressive field in a simple compressive strut is the prismatic or parallel stress field.

In the strut-and-tie models, the struts and ties condense the real stress fields' by-products through straight lines and concentrate their curvature in nodes. A force paths method has been provided by Schlaich *et al* [S2] to orient the struts and ties. The basic idea of the load-path method is to balance the by-products of the stress diagram action at one border in a region with their counterparts at the other border, as well as any loads applied within the region. Load paths are then used to connect the forces from the opposite borders, with diversions made as necessary at nodes by means of transverse ties and struts.

Figure 2.3.3 Types of compression Stress fields

An elastic analysis using the finite element method (FEM) can help to identify the load paths of a structure to develop appropriate strut-and-tie model. After the elastic stresses and principal stress directions are established through the FEM analysis, the directions of the struts can then be taken in accordance with the mean direction of the principal compressive stresses, or the more important struts and ties can be located at the centers or at the points of gravity of the corresponding stress diagrams.

2.3.4 Strength of struts

The compressive strength of concrete is affected by the presence of transverse stresses. In the case of biaxial or tri-axial compression, the compressive strength can be substantially enhanced. On the other hand, a transverse tensile stress field reduces the compressive strength of the strut. The effective compressive strength of the concrete is also reduced by the presence of cracks and inclined reinforcing bars. In general, it is chosen as some portion of concrete cylinder compressive strength:

$$f'_{ce} = \xi f'_c \tag{2-1}$$

where ξ = the effective factor which is less than unit

f'_{ce} = the effective strength of the concrete struts

f'_c = concrete cylinder compressive strength

Considerable research has been conducted in an effort to determine the limiting concrete compressive stress of struts. Some researchers and ACI Committee suggested constant values for the effective factor under different conditions as listed in **Table 2.1**.

Table 2.1 Effective factor ξ of the strength of the concrete strut

Proposed by	Description of struts	ξ
Schlaich <i>et al</i> [S2]	Undisturbed and uniaxial state of compression (e.g., prism)	0.85
	Cracks with normal width parallel to the strut	0.68
	Skew cracks with normal width or skew reinforcement	0.51
	Skew cracks with extraordinary crack width	0.34
MacGregor [M1]	Uncracked uniaxially stressed strut or fields	v_2^1
	Struts cracked longitudinally due to bottle shaped stress fields with transverse reinforcement	$0.80 v_2$
	Struts cracked longitudinally due to bottle shaped stress fields without transverse reinforcement	$0.65 v_2$
	Struts in cracked zone with transverse tensions from transverse reinforcement	$0.60 v_2$
	Severely cracked webs of slender beams with $\theta = 30^\circ$	$0.30 v_2$
	Severely cracked webs of slender beams with $\theta = 45^\circ$	$0.55 v_2$
Alshergirir and Ramirez [A8]	Moderately confined diagonal struts extending from point load to support with shear span to depth ratio < 2.0	0.85
	Struts forming arch mechanism	0.75
	Arch members in prestressed beams and fan compression members	0.50
	Undisturbed and highly stressed struts or fields	0.95
Marti [M5]	All type of struts	0.6
ACI 318 [A1]	Struts of uniform cross-sectional area	0.85
	Bottle shaped struts with adequate reinforcement	0.64
	Bottle shaped struts without reinforcement	$0.6 \lambda^2$

¹. $v_2 = 0.5 + 1.25 / \sqrt{f'_c}$ in MPa after Bergmeister *et al* [B4]

². $\lambda = 1.0$ for normal weight concrete, 0.85 for sand-light weight concrete and 0.75 for all light weight concrete

Other research has suggested that the effective strengths of the concrete struts were related to some other factors such as the strength and strain of the concrete as well as structural geometry. Based on the test of beams, Nielsen *et al* [N3] proposed an empirical equation for the effective strength of concrete struts in beam webs:

$$f_{ce} = (0.7 - \frac{f'_c}{200}) f'_c \quad (2-2)$$

where the unit is in *MPa*.

The Canadian Concrete Code (1984) [C1] recommends that the effective concrete compressive strength of a concrete strut be taken as:

$$f_{ce} = \frac{\lambda \phi_c f'_c}{0.8 + 170 \epsilon_1} \geq \lambda \phi_c f'_c \quad (2-3)$$

where λ = factor which depends on the concrete density, namely

1.0 for normal weight concrete,

0.85 for semi-light weight concrete, and

0.75 for light weight concrete.

ϕ_c = material factor for concrete=0.6

ϵ_1 = principal tensile strain at right angles to the direction of the strut.

The parameter ϵ_1 accounts for the fact that the effective strength of a strut decreases when the tensile strains produced by the action of transverse reinforcement run perpendicular to the direction of the compressive force. **Figure 2.3.4 (a)** shows a compression field with some reinforcement in the x direction. The strut is inclined at an angle θ to the reinforcement. ϵ_1 is the principal tensile strain at right angles to the strut and ϵ_2 is the principal compressive strain in the direction of the strut. The strain ϵ_1 is determined from the strain compatibility in the region where reinforcement action as tensile ties crossed the compressive strut. From the Mohr's circle of strain of **Figure 2.3.4 (b)** ϵ_1 is given by the expression:

$$\epsilon_1 = \epsilon_x + \frac{\epsilon_x + \epsilon_2}{\tan^2 \theta} \quad (2-4)$$

If ϵ_x is replaced by the strain ϵ_s in the tension tie, and assuming that the compressive strain in the concrete strut when its strength is reached is 0.002, then

$$\varepsilon_1 = \varepsilon_s + \frac{\varepsilon_s + 0.002}{\tan^2 \theta} \quad (2-5)$$

where ε_s may be the yield strain of the reinforcement.

Figure 2.3.4 Strains in the cracked compressive concrete strut

Based on experimental results of testing large panels subjected to uniform stress and strain fields, Hsu [H6] derived an equation to determine the concrete effective factor:

$$\xi = \frac{0.9}{\sqrt{1 + 600\xi_r}} \quad (2-6)$$

where ξ_r = the tensile strain.

Effective compressive strength formulae proposed by Muttoni, Schwartz & Thurlimann [M13] are as follows:

For uncracked concrete with transverse compressive stress and α is defined below:

$$f_{ce} = 20\alpha + 4\sigma_1 \quad (2-7)$$

where σ_1 = the transverse compressive stress;

The values have been derived by treating 20 MPa as a reference stress, and the term α is defined in terms of the ratio of f_{ce} to the reference stress value:

$$\alpha = [f'_c / 20]^{2/3} \quad (2-8)$$

For uncracked uniaxial compression:

$$f_{ce} = 20\alpha \quad (2-9)$$

For concrete with cracks parallel to the compressive stress, or laterally imposed tensile

strains:

$$f_{ce} = 0.8 \times 20\alpha \quad (2-10)$$

For concrete with cracks parallel to the compressive stress, or diagonally imposed tensile strains:

$$f_{ce} = 0.6 \times 20\alpha \quad (2-11)$$

The above expressions apply for concrete with f'_c not less than 20 MPa.

Foster and Gilbert [F4] suggested that the effective factor is also related to the span-to-depth ratio (a/d). They propose the following equation of the effective factor for concrete strength up to 100 MPa.

$$\xi = \frac{1}{1.14 + (0.64 + f'_c/470)(a/d)^2} \leq 0.85 \quad (2-12)$$

They [F4] further simplified it leading to the modified Collins & Mitchell equation:

$$\xi = \frac{1}{1.14 + 0.75(a/d)^2} \leq 0.85 \quad (2-13)$$

Based on the results of a series of nonlinear finite element analyses, Warwick & Foster [W5] proposed the equation:

$$\text{for } a/d < 2, \quad \xi = 1.25 - \frac{f'_c}{500} - 0.72\left(\frac{a}{d}\right) + 0.18\left(\frac{a}{d}\right)^2 \leq 0.85 \quad (2-14)$$

$$\text{for } a/d \geq 2, \quad \xi = 0.53 - \frac{f'_c}{500} \quad (2-15)$$

2.3.5 Nodes

According to ACI 318 [A1], a node is the point in a joint of a strut-and-tie model where the axes of the struts, ties and concentrated forces acting on the joint intersect, while a nodal zone is the volume of concrete around a node that is assumed to transfer strut-and-tie forces through the node. The ACI 318 code [A1] also defines an extended nodal zone to consider the anchorage of the reinforcement joining it. An extended nodal zone is that portion of a member bounded by the intersection of the effective strut width, w_s and the effective tie width, w_t . Nodes may be classified according to the number of entering

elements and their types (whether struts or ties). For a three-element node, the possible combinations are CCC, CTC, TCT, and TTT. The C refers to a compression strut and the T to a tension tie. **Figure 2.3.5** presented the nodal zone and extended nodal zone of a CCT node and a TCT node. Schlaich [S3] has also given detailed guidance for the most common configuration of nodes. The ideal type of node is CCC because the biaxial state of stress is produced within it. The other types especially CTC and TCT can be transformed into CCC by the way that the ties pull the node from behind. Type CCC or transformed CCC nodes should be preferred in any seismic design.

If the strut widths are chosen so that the stresses in all struts are equal to $-\sigma$, the principal stresses in the node will be $\sigma_1 = \sigma_2 = -\sigma$ with the sides of the node are perpendicular to the struts and thus a hydrostatic nodal zone is formed. If the stresses in the struts are unequal, the state of stress in the node can be found by using Mohr's circle technique proposed by Marti [M5]. The strength of concrete in the nodal zones depends on many factors, such as the concrete's confinement by reactions, the compression struts and hoop reinforcement, and the effect of tensile strains resulting from the steel tension ties anchored within the node. The permissible stresses in nodal zones proposed by various researchers are listed in **Table 2.2** [A2].

Figure 2.3.5 Nodal zone and extended nodal zones [A1]

Two key points in the design of nodes are to ensure that the compressive stresses within the node are less than the allowable limits, and to ensure appropriate anchorage of the ties at the nodes. The following sequence is suggested for detailing or designing a node:

- 1) choose the geometry of the node;
- 2) check that the concrete stresses are not excessive; and
- 3) provide adequate anchorage for any bars, if necessary by anchor plates.

Table 2.2 Permissible stresses in nodal zones

Proposed by	Condition of the node	Permissible stress
Schlaich <i>et al</i> [S3]	CCC nodes	$0.85 f'_c$
	Nodes where reinforcement is anchored in or crossing the node	$0.68 f'_c$
MacGregor [M1]	CCC nodes	$\nu_2(0.85) f'_c$ ¹
	CCT nodes	$\nu_2(0.65) f'_c$
	CTT and TTT nodes	$\nu_2(0.50) f'_c$
Collins and Mitchell [C6]	CCC nodes	$0.85 \phi f'_c$ ²
	CCT nodes	$0.75 \phi f'_c$ ²
	CTT and TTT nodes	$0.60 \phi f'_c$ ²
Marti [M5]	Any nodes	$0.60 f'_c$
Bergmeister <i>et al</i> [B4]	Unconfined nodes	$0.80 f'_c$ for $f'_c \leq 28 \text{ MPa}$
	Unconfined nodes	$(0.9 - 0.25 f'_c / 69)$ for $28 \leq f'_c \leq 69 \text{ MPa}$
	Unconfined nodes	$0.65 f'_c$ for $f'_c \geq 69 \text{ MPa}$
	Confined nodes	See Ref. [B4] for various equations with a maximum limit of $2.5 f'_c$
	Traiaxially confined nodes	$2.5 f'_c$
ACI 318 [A1]	Nodes bounded by struts or bearing areas or both	$0.85 f'_c$
	CCT nodes	$0.68 f'_c$
	CTT or TTT nodes	$0.51 f'_c$

¹. $\nu_2 = 0.5 + 1.25 / \sqrt{f'_c}$ in MPa after Bergmeister *et al* [B4]

². ϕ is the strength reduction factor, for bearing on concrete, $\phi = 0.70$

As stated in this and previous sections, the effective concrete strength of strut and node may vary with the crack, confinement, and boundary conditions etc. A comparison of the effective strength of strut and node proposed by different researchers and design codes has been conducted by Su and Chandler [S6]. Based on this, design formulae for the strength of strut and node are proposed, which take into account (i) the orientation of the strut and tie; (ii) the brittle effects as the strength of concrete increases; (iii) the strain

state of both concrete and steel; and (iv) the stress stat of the node boundary. In this study, considering no detail guideline for strength of the strut and node in a wall has been provided, the effective strength proposed by Schlaich et al [S3] were applied.

2.3.6 Modelling procedure

The overall design using strut-and-tie models can be generalised by the following steps:

- 1) break the structure up into B-regions and D-regions;
- 2) for each D-region, consider the alternative strut-and-tie models;
- 3) identify which model is appropriate;
- 4) for this model, detail the struts, ties and nodes for strength; and
- 5) investigate the requirements for secondary reinforcement to control cracking.

2.4 Capacity Design

The characteristics of the earthquake ground motions that may occur at a given site cannot be predicted with certainty and it is difficult to evaluate all aspects of the complete behaviour of a complex structure when subjected to a severe earthquake. Nevertheless it is possible to design the structure so that it has features that will ensure the most desirable behaviour. The rational approach for achieving this aim in design for earthquake resistance is to choose the most suitable mechanism of post-elastic deformation for the structure and to ensure by appropriate design procedures that yielding will occur only in the chosen manner during a severe earthquake. “Capacity design” has been proposed to fulfill this aim.

The procedures for the application of capacity design to ductile structures, which may be subjected to large earthquakes, have been developed primarily in New Zealand for about thirty years [P3, P7]. In the capacity design of structures, appropriate regions of the primary lateral earthquake force resisting structural system are chosen and suitably designed and detailed to provide adequate strength and ductility for a severe earthquake. All other regions of the structural system and other possible failure modes are then provided with sufficient strength to ensure that the chosen means for achieving ductility can be maintained throughout the post-elastic deformations that may occur. This is the basic concept underlying the capacity design philosophy. The critical regions of this structural system often termed “plastic hinges” are detailed for inelastic flexural action.

Shear failure can be inhibited in the region by a suitable strength differential.

The capacity design approach requires consideration at three levels of member strength, namely, design strength S_u , nominal strength S_n , and overstrength S_o . Design strength is the nominal strength multiplied by the appropriate strength reduction factor ϕ . Nominal strength is the theoretical strength calculated by using the member's dimensions, which can be detailed as the lower characteristic yield strength of the steel reinforcement and the specified concrete compressive strength. Overstrength is the theoretical potential maximum strength that can be calculated using the maximum likely overstrength of the steel reinforcement, the increased concrete strength due to confinement, and the reinforcement area including any additional reinforcement placed for construction and otherwise unaccounted for in calculations.

Potential plastic hinge regions within the structure are designed to have dependable flexural strengths as close as practicable to the design strength S_u . Subsequently these regions are carefully detailed to ensure that estimated ductility demands in these regions could be reliably accommodated. This is achieved primarily by close-spaced and well-anchored transverse reinforcement. Undesirable modes of inelastic deformation, such as those may originate from shear or anchorage failures and instability within members containing plastic hinges, are inhibited by ensuring that the strengths of these modes exceed the capacities of the plastic hinges at overstrength. Potentially brittle regions, or those components not suited for stable energy dissipation, are protected by ensuring that their strength exceeds the demands originating from the overstrength of the plastic hinges. Therefore, these regions are designed to remain elastic irrespective of the intensity of the ground shaking or the magnitudes of any inelastic deformations that may occur. This approach enables traditional or conventional detailing of these elements, such as used for structures designed to resist only gravity loads and wind forces, to be employed during construction.

The capacity design of such structures can be performed by following these major steps:

1. Choose a kinematically admissible plastic mechanism, which can ensure that the necessary overall displacement ductility can be developed with the smallest inelastic rotational demands in the plastic hinges
2. Determine the regions for energy dissipation with a relatively high degree of precision.

3. Design the parts of the structure intended to remain elastic in all events so as to ensure that no inelastic deformations occur under maximum feasible actions corresponding to overstrength in the plastic hinges.
4. Make a clear distinction with respect to the nature and quality of detailing for potentially plastic regions and those that are to remain elastic in all events.

2.5 Summary

A literature review of the performance of reinforced concrete walls with irregular openings, the strut-and-tie models and the capacity design procedures has been presented in this chapter.

Current research on the structural walls with openings has demonstrated that properly designed and detailed walls can exhibit adequate ductility under reversed cyclic loadings. The strut-and-tie models and capacity design can provide guidelines in the design of walls with openings.

The strut-and-tie models, which are the generalization of truss analogy and have been developed for more than a century, still provide great potential in the study of the behaviour of cracked reinforced concrete. The standard truss models developed on B-regions and more precise models based on the equilibrium, compatibility and constitutive law, have been proposed and successfully utilised in practical designs. Currently, strut-and-tie models are regarded as powerful tools in understanding the behaviour of D-regions. Based on the models, the D-regions can be designed and detailed properly.

Walls with irregular openings are non-flexural members because complicated stress states will occur when they are subjected to lateral loadings. Therefore, the applications of strut-and-tie models and capacity design procedures on the seismic design for this kind of walls have become a focus in this study.

CHAPTER THREE

NONLINEAR FINITE ELEMENT ANALYSIS OF WALLS WITH OPENINGS

3.1 Introduction

Nowadays the finite element method (FEM) has become an important and indispensable part of engineering analysis. With the development of the nonlinear analysis procedures, especially in the nonlinear constitutive laws of materials, the nonlinear behaviour of reinforced concrete structures can be reliably simulated. After quantifying the material-based damage, it is possible to make an accurate judgment on the serviceability or function of a structure during or after an earthquake so that the performance-based design criteria can be fulfilled with the aid of nonlinear finite element analysis [M3]. The continuing development on the computer technology also makes a complicated nonlinear finite element analysis on reinforced concrete structures become possible and efficient. It is of great benefit to applying the nonlinear finite element analysis as a performance check on the design of a structure.

Taking advantages of the nonlinear finite element analysis to simulate a laboratory experiment can provide a lot of information toward understanding the structural behaviour. For example, the load path in a structure can be visualised by the stress contours. Besides, the parameters, which have not been considered in the experiment, could be applied in the nonlinear finite element simulation. Therefore, in this study, a two-dimensional (2D) simulation on the Yanez's experiment [Y2] and Ali's experiment [A4] was performed using program WCOMD [M3, O1] to further understand the behaviour of a wall with openings. A three-dimensional (3D) nonlinear finite element analysis on wall using program DIANA 8.1 [W6] is also introduced in this chapter. It will be used to simulate the experimental programme on flanged walls with openings which would be introduced in the following chapters. Program WCOMD [M3, O1] will also be adopted to carry out a parametric study as stated in Chapter 6.

It is important to choose a reliable and efficient programme to simulate the reinforced concrete structural behaviour to ensure that a viable result can be obtained. Programs

WCOMD and DIANA have been proved to fulfil this requirement. Program WCOMD (Concrete Model for in-plane Wall under Dynamic loads) [M3, O1] has been developed at the University of Tokyo for about twenty years, and has been verified to be reliable and efficient in simulating the behaviour of RC structures under static and dynamic loadings. The details of development and verification of this program were well documented on Okamura and Maekawa's books [M3, O1]. Program DIANA 8.1 (Displacement method Analyser, version 8.1) has been under development at the TNO Building and Construction Research in the Netherlands since 1972. It is a high quality finite element analysis program developed for a wide range of civil engineering applications, with particular strengths in concrete, seismic and geomechanics. Extensive materials, elements and procedure libraries based on advanced database techniques are provided in this program. It can perform linear and non-linear analysis in full 2D and 3D environments. Engineers worldwide have successfully applied DIANA to their works on bridge and building designs, dams, offshore platforms, road and rail designs, tunnelling, nuclear plant and earthquake analyses *etc* [W6].

3.2 Main Aspects on Modelling

Before the detailed modelling method using programs WCOMD and DIANA8.1 are introduced, the main aspects on modelling a reinforced concrete structure are first reviewed in this section.

3.2.1 Modelling concrete

In a two-dimensional nonlinear finite element analysis of wall, eight-node quadrilateral isoparametric plane stress elements are generally used to model concrete. In a three-dimensional analysis, twenty-node isoparametric solid brick elements are applied. Both elements are based on quadratic interpolation. The 3×3 Gauss integration scheme is applied to plane stress element in WCOMD and the $3 \times 3 \times 3$ Gauss integration scheme is used over the solid brick element.

The linearly interpolated isoparametric elements, such as eight-node isoparametric solid brick element, are not recommended to be used in the nonlinear analysis because the shortcomings of these elements, like parasitic shear, cannot be easily dealt with in the nonlinear analysis [W6].

3.2.2 Modelling reinforcement

Two techniques are generally used to model reinforcement in a reinforced concrete structure. The first method is called embedded reinforcement or smeared reinforcement technique. As shown by the name, the reinforcement is embedded in the structural element, i.e. the so-called “mother element”. The stiffness of the “mother element” is then modified based on the pattern of embedded reinforcement. Reinforcement does not have its own degrees of freedom. Its strains are computed from the displacement field of the mother element, so a perfect bond is assumed between the reinforcement and the surrounding material. The second method is using separate elements to model reinforcement, for example truss elements are often applied. In some cases, beam elements are used because their restricted rotation of the beam end nodes can simulate the behaviour of reinforcement more realistically, such as when the confinement effect of the reinforcement is to be considered. The reinforcement is connected with the surrounding structural material in the nodes. An embedded reinforcement technique involves less computational demand and allows the line of the reinforcement to deviate from the lines of the mesh, but the bond-slip relationship can only be implicitly incorporated into the constitutive laws of cracked concrete and reinforcing bars rather than be reflected separately. The second technique allows for the investigation of the bond-slip behaviour of reinforcement by adding interface elements between the reinforcement and the concrete. However, high computational cost is demanded in this technique and it requires that the lines of reinforcement are located at the sides of the structural elements. It is not easy to model diagonal reinforcement in this way whereas they can be easily modelled as embedded reinforcement.

In programme WCOMD, an embedded reinforcement technique is applied. Both schemes can be used in programme DIANA 8.1. In this study, reinforcement is generally modelled as truss elements when programme DIANA 8.1 is applied.

3.2.3 Modelling discontinuities

Large deformations are often observed at the discontinuities of a structure, such as the pulling out of reinforcing bars from a thicker element and shear slipping along the joint between two elements. The structural deformations may be underestimated if the effects of discontinuities are overlooked. Interface elements are applied to the discontinuities to accommodate this effect. Interface behaviour is generally described in terms of a

relationship between the normal and shear tractions and the normal and shear relative displacements across the interface. **Figure 3.2.1** shows the local discontinuities where the interface elements are required. To cope with the different local discontinuities, joint element which is a kind of interface elements is provided in WCOMD, and DIANA8.1 provides a group of structural interface elements, including nodal, line and plane interface elements.

Figure 3.2.1 Discrete cracks at joints

3.2.4 Constitutive models

Nonlinear constitutive laws for materials can have significant influence on the nonlinear finite element analysis results. A valid simulation result should be based on reliable constitutive models of materials. The nonlinear behaviour of a reinforced concrete element is primarily caused by cracking in the concrete, plasticity of the reinforcement and bond-slip between the concrete and reinforcement, which should be reflected in the constitutive laws of materials.

When reinforcement is modelled as separate elements, the constitutive model for bare bar can be applied. It is generally controlled by four parameters, namely, Young's modulus E_s , hardening modulus E_{sh} , yield strength f_y and ultimate strength f_u . However, when an embedded reinforcement technique is applied, the bond effect should be considered in the constitutive law of reinforcement.

The constitutive model of concrete in a structure can be complicated because of the complexity of concrete material. Concrete is an anisotropic and non-homogeneous material, can sustain high compression but carry very low tension, and often cracks under low tensile stress. It is a kind of quasi-brittle materials. Presently, concrete constitutive laws have been developed within two categories. They are macroscopic models and

microscopic models. In a macroscopic model, the complex stress-strain relationships of microstructures are expressed as average stress-strain relationships on a control volume, while in a microscopic model, the stress-strain relationship is described at a micro level. Microscopic models are more promising than macroscopic models from a physical point of view, but they often involve high computational cost and so macroscopic models are often used in practical application [O2]. The following behavioural aspects of response are generally needed to cover in a proper concrete model of concrete. They are (1) compressive behaviour, (2) compression hardening and softening behaviour (3) tensile behaviour, (4) tension stiffening and softening behaviour, and (5) cracking behaviour.

A smeared crack approach, also known as crack band theory [B2], is general applied to the macroscopic models of concrete. According to this approach, the generation and propagation of an individual crack is smeared into a finite region, and then the average stress-strain relationship is adopted. By way of determining a crack direction, this approach can be divided into the rotating crack approach and the fixed crack approach. The primary assumption of the rotating crack approach is the coaxial stress-strain concept, in which the stress-strain relationships are evaluated in the principal directions of the strain vector. Accordingly, the crack direction coincides with the principal direction of average strain and can be changed or rotated following the stress condition. Because no shear stress appears in continually updated principal planes, shear transfer model is not needed in this approach. The fixed crack approach assumes that the crack is fixed once it is generated. The stress-strain relationships are then evaluated on the fixed coordinate system upon the crack. In this model, the shear transfer model is needed but the principal directions of stress and strain need not coincide with each other. Compared with rotating crack model, the fixed crack model is geometrically closer to the real crack state in the concrete, plus strong anisotropy and non-orthogonal multi-cracking interaction in shear can be taken into account by this approach. It shows a wider range of applicability than the rotating crack model [M3]. However, in terms of simplicity and reasonable accuracy, the rotating smeared crack model is often applied.

3.2.5 Nonlinear iteration methods

In nonlinear FEM analysis, Newton-Raphson iteration strategies are often used. These solution strategies can generally be divided into two subclasses: the Regular Newton-Raphson methods and the Modified Newton-Raphson methods. In a Newton-Raphson method, the stiffness matrix represents the tangential stiffness of the structure. The

difference between the Regular and the Modified Newton-Raphson method is the point at which the stiffness matrix is calculated, as shown in **Figures 3.2.2 (a)-(b)**.

(a) Regular Newton-Raphson iteration method

(b) Modified Newton-Raphson iteration method

(c) Linear and Constant Stiffness iteration method

Figure 3.2.2 Nonlinear iteration methods [W6]

In the Regular Newton-Raphson iteration method, the tangential stiffness matrix is recalculated in each step of iteration, and as a result the method yields a quadratic convergence characteristic, so only a few iterations are needed to converge to the final solution. However, it is time consuming to set up and decompose the stiffness matrix at every step of iteration and the quadratic convergence is only guaranteed if a correct stiffness matrix is used. If the initial prediction is far from the final solution, the method easily fails because of divergence. Therefore, this method can be applied for moderate nonlinear behaviour where light damage increases at every step. Otherwise the Modified Newton-Raphson method can be used. This method only evaluates the stiffness relation at the start of the increment, which ensures that the prediction is always based on a converged equilibrium state. Compared with the Regular Newton-Raphson method, the Modified Newton-Raphson converges more slowly to equilibrium but for every step of iteration it is not necessary to set up a new stiffness matrix.

For highly nonlinear situations where Newton-Raphson methods do not converge anymore, the Linear and Constant Stiffness iteration methods can be used (**Figure 3.2.2c**). The Linear Stiffness solution uses the linear stiffness matrix all the time, so it potentially has the slowest convergence, but it costs the least time per iteration since the stiffness matrix needs to be set up and decomposed only once, also it is usually very robust. The Constant Stiffness method uses the stiffness matrix left behind by the previous increment. If the Constant Stiffness iteration is used since the first increment, this method equals the Linear Stiffness method. It can also be applied if the Newton-Raphson methods fail after a number of successful increments.

3.3 2D Modelling Using WCOMD

3.3.1 General

Developed and verified by the University of Tokyo, programme WCOMD can perform path-dependent two-dimensional static and dynamic non-linear analysis of various reinforced concrete structures accurately, including cracks [M3, O1]. The maximum response displacement or strain information and the residue after the calculation, can also be obtained. The safety evaluation of structures or the investigation of damage levels can also be carried out as a result of the output.

The smeared crack model and the joint element model developed by the University of Tokyo for cracked reinforced concrete are used in this program. The smeared crack model is suitable for members with distributed two-directional reinforcement. For the local discontinuous regions, the joint element model is applied. This program combining the advantages of two types of elements can predicted the behaviour of reinforced concrete structures subjected to reversed cyclic loadings, especially for reinforced concrete walls [M3, O1].

Four plate type elements, including RC Plate, Plain Concrete Plate, Elastic Plate and Soil Plate, are available in this program. Generally, eight node quadrilateral isoparametric elements, based on quadratic interpolation and the 3×3 Gauss integration scheme, are applied to model RC and plain concrete plates. An embedded reinforcement technique is used to deal with reinforcement. The bond effect is considered in the constitutive law developed for RC plates. Newton-Raphson iteration methods are adopted for solutions.

3.3.2 Constitutive law

The constitutive law of cracked concrete in WCOMD is developed based on the fixed smeared crack approach. It consists of a compression model, a tension stiffening model, and a shear transfer model [M3, O1]. The modelling of the reinforcing bar in this program takes the effect of bonding between the bar and concrete into account. For modelling local discontinuities, the joint element, which is a one-dimensional interface element with zero thickness, is used. The details and verification of the development are well stated in Maekawa and Okamura's books [M3, O1]. Only a concise review is presented here.

Figure 3.3.1 shows the EPF (elasto-plastic fracture) model for concrete before cracking. The uncracked concrete is regarded as a material of plasticity and continuum fracture because permanent deformation and the loss of elastic strain energy absorption occur in it. A softened unloading path is used in this model.

Similar to the compression field theory [C6, V2], the cracked concrete is assumed to be uniaxially compressed and is softened by the tensile strain in the orthogonal direction.

Figure 3.3.2 shows the model for cracked concrete.

Figures 3.3.3 (a) and (b) present the tension stiffening and tension softening models for reinforced concrete and plain concrete (PL). These models are expressed as

$$\sigma = f_t \cdot (\epsilon_{tu} / \epsilon)^c \quad (3-1)$$

where σ = average tensile stress of concrete
 ε = average tensile strain of concrete
 f_t = uniaxial tensile strength of concrete
 C = stiffening/softening factor

Figure 3.3.1 Concrete compression model before cracking [M2]

Figure 3.3.2 Uniaxial concrete compression model parallel to cracking [M2]

Figure 3.3.3 Concrete tension model [M2]

In reinforced concrete, the tension stiffening factor C is 0.4 for deformed bars and 0.2 for welded wire mesh. In plain concrete, the softening factor C must be determined from fracture energy G_f , which is the energy consumed per unit crack propagation. It is dependent on the size of element as shown in **Figure 3.3.3 (b)**.

The unloading-reloading scheme of concrete is modelled by a coupled compression-tension path-dependent model as shown in **Figure 3.3.4**. It is a direct combination of the path of the compression, tension and re-contact stress models, in which the re-contact model is used to consider the amount of compressive stress transferred by a crack before it is perfectly closed.

Figure 3.3.5 demonstrates the shear transfer model for cracked concrete. It is based on the contact density theory by idealising a rough crack as a set of numerous infinitesimal contact units having various inclinations.

The reinforcing bar model is presented in **Figure 3.3.6**. The influence of the bond effect is also taken into account in this model by assuming that the distribution of bar stress between two adjacent cracks follows a cosine curve.

In order to deal with size effect existing in a failure governed by the energy released, such as failure induced by cracking, a zoning method proposed by An *et al* [A9, M3]] is adopted in this program. According to this method, the overall concrete of a structure can be divided into plain zones (PL) and RC zones. In PL zones, the constitutive laws for plain concrete are applied while in RC zones, the constitutive laws for reinforced concrete must be applied. The size of the RC zone is governed by the crack control ability of the reinforcement. The maximum area of the RC zone for a single bar is the certain area when the concrete in this area cracks and the released tensile force can just load the bar to yield. Assuming the RC zone is square, the maximum height of the RC zone $h_{\max}$ can be determined by

$$h_{\max} = \frac{\sqrt{\pi}}{2} \cdot d_b \cdot \sqrt{\frac{f_y}{f_t}} \quad (3-2)$$

where d_b = diameter of the reinforcing bar

f_y = yield strength of bar

Basically, the height of RC zone is determined by the diameter of each bar, but it is also related to the bar arrangement as shown in **Figure 3.3.7**. It is important to distinguish the

RC zones from the PL zones during modelling in order to obtain reliable simulation results. However, in some cases, the PL and RC zones cannot be figured out in two-dimensional model or the height of the RC zone is too small to be applied as an element height as shown in **Figure 3.3.8**, a mixed element containing both PL and RC zones can be used. The constitutive law of the mixed element can be obtained by averaging the responses of PL and RC zones in the total area of these two zones.

Based on the definition of RC zone, the effective reinforcement ratio ρ_{eff} can be expressed as

$$\rho_{eff} = A_{st} / A_{RC} \quad (3-2)$$

where A_{st} = area of reinforcement effectively bonded to concrete in tension

A_{RC} = area of RC zone surrounding the reinforcement

The effective reinforcement ratio ρ_{eff} is also related to the bond-slip behaviour in reinforced concrete. Generally, the smaller the ρ_{eff} is, the lower the average yield level of reinforcement would be.

3.3.3 Pre-processing programme UC-WIN/MESH

Program UC-WIN/MESH developed by company FORUM8 is the pre-processor of WCOMD. It is a tool for creating and editing mesh data used by WCOMD for finite element analysis. When editing the mesh, the following general principles should be followed to define element size and shape:

- 1) The elements with different physical properties must be placed in different meshes.
- 2) The mesh should be divided finely in stress localisation areas. Larger elements would be adequate in low stress areas.
- 3) It is recommended that the shape of the plates were defined as rectangular except for the corners of structures.
- 4) In reinforced concrete plates, meshes must be defined depending on the density of the reinforcement ratio. Furthermore it is expected that the centre of gravity of a mesh would coincide with that of the reinforcing bars, especially for main reinforcing bars.

Figure 3.3.4 Uniaxial coupled tension-compression model [M3]

Figure 3.3.5 Shear transfer model for one-directional crack [M3]

Figure 3.3.6 Reinforcing bar model [M3]

Figure 3.3.7 Size of the RC zone [M3]

Figure 3.3.8 Mixed element containing both PL and RC zones [M3]

- 5) It is recommended that the side length of a RC plate should be approximately within the range of a few centimetres up to as much as one hundred centimetres. If the element size is too large the analysis may become unstable and will be recognised by a zigzag shape in the result charts, such as the load-displacement relationship.

3.4 3D Modelling Using DIANA 8.1

3.4.1 General

Compared with WCOMD, DIANA8.1 possesses more element types and material properties and can perform 3D analysis, so it can be applied to a wider range of applications in civil engineering. In this study, it is mainly applied to perform the 3D nonlinear analysis of the flanged walls with openings. The results will be shown in the following chapters.

The twenty-node isoparametric solid brick elements based on quadratic interpolation and $3 \times 3 \times 3$ Gauss integration scheme are applied to model concrete in 3D nonlinear finite element analysis. The reinforcement generally is modelled as separate truss elements. They are arranged at the side of the brick elements. The embedded reinforcement technique is used only in the case of diagonal reinforcement being applied. The Regular Newton-Raphson iteration method is applied for the solution.

3.4.2 Constitutive law

In 3D analysis, concrete constitutive models can be formed by combining the compressive yield surface of an established 3D model with a tension cut-off. The compressive yield surface is built based on the theory of plasticity. It involves the application of a yield criterion, a flow rule, and hardening rules. Von Mises and Drucker-Pager yield criteria [C5, W6] are often used to model concrete. There are two tension cut-off models for concrete, namely, constant and linear cut-off models. Once the principal tensile stress in the concrete exceeds the maximum tensile stress indicated by the tension cut-off model, a crack arises. However, these models cannot accurately describe the softened behaviour of concrete, which is associated with the fracture inside concrete.

3D concrete constitutive models can also be created based on the total strain concept. They are called “total strain crack models”. As stated before, they can be further divided into total strain rotating crack models and total strain fixed crack models. These models describe the concrete behaviour by one tension and compression stress-strain relationship, which can be obtained in a uniaxial stress state. These models are very suitable for modelling cracked concrete and will be adopted in this study.

In a total strain model, concrete compression behaviour can be modelled by seven curves pre-defined by DIANA8.1 as presented in **Figure 3.4.1**. They include the elastic curve, the constant curve, the brittle curve, the multi-linear curve, the linear hardening and the saturation hardening curves, the Thorenfeldt curve, and the parabolic curve. The elastic curve generally is not used for nonlinear analysis. Only the Thorenfeldt curve and parabolic curve can consider the softening behaviour of concrete. The parabolic curve is based on the concrete compressive fracture energy G_c by the definition of the crack band width h of the element. In DIANA 8.1, the default crack band width h is $\sqrt[3]{V}$ for solid

elements, where V is the volume of the element. The compressive fracture energy G_c can be approximated as 150 times larger than concrete fracture energy G_f [O2].

Figure 3.4.1 Concrete compression models [W6]

DIANA8.1 pre-defined seven tension curves to model the tension behaviour of concrete as plotted in **Figure 3.4.2**. If elastic behaviour is applied, no crack would open, so it is not used in nonlinear analysis. In the ideal model, no tension softening behaviour is presented. In a brittle model, it is assumed that concrete abruptly loses tensile strength once it cracks. The tension behaviour of plain concrete or concrete with low reinforcement is close to this model. For properly reinforced concrete, the model with a softening branch is more suitable. There are four softening models as shown in the figure, including linear, multi-linear and nonlinear softening models. All these softening curves are based on fracture energy G_f by the definition of the crack band width of the element like the parabolic compressive model. Nonlinear models include exponential and Hordijk models.

The shear behaviour of concrete is necessary in the constitutive model based on the fixed crack concept. The shear stiffness of a material is usually reduced after cracking. This reduction is generally known as shear retention. In DIANA8.1, a constant shear stiffness reduction model is defined. The shear modulus G is reduced to shear modulus after cracking G^{cr} by the equation

$$G^{cr} = \beta \cdot G \quad (3-3)$$

where β = shear retention factor

It varies from 1 for uncracked concrete to 0 for completely cracked concrete. The shear stiffness may decay with the crack propagation, so a variable shear retention model may be more suitable. For a total strain model, the variable shear retention model can be approximated as a multi-linear model by inputting a group of data on an established variable shear retention curve.

The reinforcement is modeled based on its uniaxial stress-strain response. Normally the Von Mises yield criterion is used. The hardening curve can be defined according to its uniaxial test curve.

Figure 3.4.2 Concrete tension softened models [W6]

3.4.3 Choice of the models

As shown in the previous section, DIANA8.1 provides several cracking, compression and tension models. In order to choose appropriate models to perform 3D analysis, a series of analyses using different models were performed and compared with the negative experimental load-displacement backbone envelope of a flanged solid wall (Specimen S-F1). The detailed experimental load-displacement response will be presented in Chapter 5.

Figure 3.4.3 plots three curves obtained from the analysis using three different cracking models. Two curves were computed using total strain models, including one fixed crack model with constant shear retention $\beta = 0.09$ and one rotation crack model. The exponential tension softening curve and the parabolic compressive curve were used in these two models. The left one was calculated using the model based on the Von Mises

yield criterion and fixed crack concept, in which the cracking was modelled as the combination of the linear tension cut-off, the brittle tension model and variable shear retention. It can be noted that the curve of the model based on the Von Mises yield criterion showed good correlation with the experimental envelop before the large plastic deformation developed in the wall but it significantly deviated from the experimental results after that. Two curves based on total strain models displayed greater stiffness at the early loading stage than the experimental curve but they got closer to it when the load increased. Compared with the curve of the fixed crack model, the curve of the rotating crack model is closer to the experimental result. Therefore, the total strain rotation crack model was applied in this study.

Figure 3.4.3 Comparisons of concrete cracking models

As stated previously, only the Thorenfeldt curve and parabolic curve can consider the softening behaviour of concrete among the seven concrete compressive curves for total strain models shown in **Figure 3.4.1**. Thus, the models using these two compressive curves were analyzed. The model based on the ideal compressive curve was also computed for comparison. The rotating crack concept and the exponential tension softening curve were applied to all models. The computed curves are shown in **Figure 3.4.4**. It indicates that the parabolic compressive model is the most suitable for analysis.

Figure 3.4.5 shows the curves calculated from the models based on different tension models, including the ideal model, the brittle model and three nonlinear tension softening

models. The rotating crack concept and the parabolic compressive curve were used in all models. **Figure 3.4.5** demonstrates that the nonlinear tension softening models give similar results. The brittle model induces a result lower than the experimental one. The ideal tension model may lead to a maximum strength much higher than the experimental result.

Figure 3.4.4 Comparisons of concrete compression models

Figure 3.4.5 Comparisons of concrete tension models

Based on these analyses, the total strain rotating crack model with the parabolic compression and the exponential tension softening behaviour is adopted for the 3D analysis. The results will be presented in Chapter 6.

3.5 2D Analysis on Wall with Openings

3.5.1 Modelling

Ten models of structural walls, including six walls (S1~S6) with low aspect ratios identical to those tested by Yanez [Y1, Y2] at the University of Canterbury and the other four slender wall models (W-1 ~ W-4) identical to the walls tested by Ali [A4 ~ A6] at the University of Michigan, were analysed with programme WCOMD. **Figures 3.5.1** and **3.5.2** show the meshes of the walls with low aspect ratios and the meshes of slender walls respectively. The meshes were built according to the methods introduced in Section 3.3. Joint elements were applied to the places of geometric discontinuities, such as the joint between wall and base beam. The details of all specimens can be found in Appendix.

The material properties and the load histories were defined according to those provided by the researchers [A4, Y2]. The load history of Yanez’s experiment was begun with three load-controlled cycles followed by the displacement-controlled cycles, including one cycle at the ductility factor of ± 1 ($\mu_{\Delta} = \pm 1$) and two cycles at each successive ductility factor. A displacement-controlled mode was applied in Ali’s experiment, in which displacement was increased according to the average drift ratios. **Table 3.1** lists the strength of the concrete used in the modelling. Two types of reinforcement were used in Specimens S1 to S6. The first one was the high yield strength steel bar (HD bars) with a yield strength of 475 MPa , the other one was the mild steel plain bar with a yield strength of 350 MPa . Three types of reinforcing were used in Specimens W-1 to W-4. #4 bars used in the boundary elements had a yield strength of 540 MPa . #2 bars were used as horizontal and vertical web reinforcement with a yield strength of 562 MPa . 3/16-in diameter wire was applied as ties in the boundary columns and the slanted reinforcement around opening corners possessing the yield strength of 620 MPa .

Table 3.1 Strength of concrete used in the model (MPa)

Specimens	S1	S2	S3	S4	S5	S6	W-1	W-2	W-3	W-4
Strength	39	28	33	31	45	26	34	32	33	34

Figure 3.5.1 Meshes of wall specimens with low aspect ratios

Figure 3.5.2 Meshes of slender wall specimens

3.5.2 Analytical results

Load-displacement responses

Figures 3.5.3 and 3.5.4 present the comparison of the analytical and experimental load-displacement responses. It can be seen that the analytical results are very close to the experimental ones, except for the difference of the strength degradation shown at the failure stage.

Crack Pattern

WCOMD can also predict the crack patterns of specimens satisfactorily. **Figure 3.5.5** shows the comparison between the analytical crack pattern and experimental crack pattern of specimen S1. Good correlation can be found.

Characteristics of stress distributions

Figures 3.5.6 to 3.5.11 illustrate the principal stress flows of wall specimens with low aspect ratios and **Figures 3.5.12 to 3.5.15** present the principal stress flows of slender wall specimens. The red lines represent the principal tensile stress and the blue ones depict the principal compressive stress. The principal stress flows induced by the loading in two directions are presented separately. For the walls with low aspect ratios, the principal stress flows at the ductility 1 ($\mu_{\Delta} = 1$) and at the ductility just before failure occurring are shown. For slender walls, the principal stress flows happening at the top drifts Δ of $\pm 19 \text{ mm}$ and the top drift just before failure are presented.

According to the characteristics of the principal stress distributions indicated by these figures, the wall with openings can be divided into different zones. A wall with irregular openings can be divided into beam zones, column zones, panel zones and nodal zone as depicted in **Figure 3.5.16 and 3.5.17**. The beam zones are the horizontal parts of the wall between the openings. The column zones are the vertical parts of the wall between the openings, including the boundary elements. The left solid parts of the wall are the panel zones. Nodal zones are the regions where column zones and beam zones intersect. As shown in **Figures 3.5.7 to 3.5.9 and Figures 3.5.13 to 3.5.15**, a diagonal principal compressive stress flow often occurs in a panel zone, while a tensile stress field often occurs in a beam or column zone. The nodes of the strut-and-tie model often lie in the nodal zones. Diagonal principal compressive stress flow can also be observed in column zones, especially when the column zones are wide, which means these zones take part in transferring a part of shear force. The wall with regular openings can be divided into panel zones and beam zones as shown in **Figure 3.5.18**. In this kind of walls, diagonal principal compressive stress flows often occur in the coupling beam and the panel zone as shown in **Figures 3.5.10 and 3.5.11**. Due to the influence of the coupled beams, the stress status in the beam zones may be different to that of the other part of wall.

3.5.3 Load paths of squat walls with irregular openings

According to Schlaich [S2], the load path of a structure is an underlying element in developing a strut-and-tie model and the load path can be achieved in accordance with the

mean direction of the principal stresses. In this part, only the load paths of squat walls with irregular openings are discussed. The load paths of other wall models can be formed in a similar way.

Based on the principal stress flows shown in **Figures 3.5.7 to 3.5.9**, the load paths for the positive and negative loading directions were developed as those depicted in **Figures 3.5.19 and 3.5.20**. **Figure 3.5.19** shows the load paths considering column contribution to resist shear, while **Figure 3.5.20** shows the load path without taking the column contribution for shear into account. With the increase of the load and loading cycle, the load paths of the walls with irregular openings changed from the former one to the latter one. Before the concrete of column zones were loaded to severe damage, such as at the loading stage $\mu_{\Delta} = \pm 1$, the column zones could transfer a part of the shear force. Referred to **Figures 3.4.7 to 3.5.9**, at this stage, diagonal principal compressive stress flows could be observed in these zones. The load paths considering column zone contribution for shear can be applied to evaluate the maximum strength of the walls. With an increase in the loading cycle, the concrete of the column were loaded to high compressive level and severe damage occurred in it, therefore, the column zones could not sustain any shear force. In this case, the load paths without considering column contribution for shear, as presented in **Figure 3.5.20**, were suitable for estimating its capacity. Normally, the structure failed after this stage was reached. The load paths presented in **Figures 3.5.19 and 3.5.20** display good correlation to the refined strut-and-tie models proposed by Yanez and the strut-and-tie models for his specimen design respectively [Y2].

The load path can also be supported by the typical principal stress states in the nodal and column zones. The stress states of four typical points (**A-D**) in the Specimens S2 and S3 shown in **Figure 3.5.19 (b)** were studied. The analysis results showed that the principal stresses including σ_1 and σ_2 at point **A** in the lower nodal zone were positive and the orientations of the principal stresses varied between $-7.5^\circ \sim 5^\circ$ and $82.5^\circ \sim 95^\circ$ respectively through each negative ductility factor ($-1 \sim -5$) when the wall was subjected to negative lateral loading (**Figure 3.5.19b**), whereas the principal stresses σ_1 and σ_2 at point **B** were also positive and the angles were within $-4.5^\circ \sim -11^\circ$ and within $-79^\circ \sim -85.5^\circ$ respectively through each positive ductility factor ($1 \sim 6$) when the wall was subjected to positive lateral loading (**Figure 3.5.19a**). This means that the nodal zone was under tensions when corresponding lateral loading was applied like the prediction by the load paths. The stress states at point **C** and **D** showed that when the

column zone was under tension, the orientation of the principal tensile stress was about 90° , which meant that axial tensions occurred in those zones. However when they were under compression, the orientations of the principal compressive stresses at points *C* or *D* changed from a sloping angle to 90° with the increase of the loading cycle. It indicated that the compressive column zones could transfer a part of the shear force at the beginning but after the concrete cracked and crushed, they could only support axial loading and could not longer transfer the shear force.

The load paths can be further proved by comparing the proportions λ of the shear force carried out by the column zones obtained from nonlinear finite element analysis and the strut-and-tie models based on the load paths. The shear force transferred by the boundary column zones under negative or positive loading can be represented by the shear force at the Sections 1-1 or 2-2 (**Figure 3.5.19a**) respectively, which can be output from finite element analysis. **Figure 3.5.21** shows the proportions (λ) of shear forces transferred by the column zones (P_c) of Specimens S2, S3 and S4 to the total lateral forces (P) in each cycle. According to the load history, cycle 4 corresponded to the displacement ductility factor $\mu_\Delta = \pm 1$, followed by the 2 cycles of at each successive ductility factor. It can be seen that the proportion reached its maximum at cycle 4 or cycle 5 then decreased as the cycles increased. After cycle 9 ($\mu_\Delta = \pm 4$), the Section 1-1 in the lower columns of Specimen S2 or S3 next to the openings, carried almost no shear force because at this ductility level the concretes there were severely cracked and crushed and the diagonal struts could not form in them, whereas the section of Specimen S4, due to its large area, continued sustaining a large proportion of the shear force thus enabling the diagonal strut in this zone to be retained.

The maximum proportion of shear force carried by the section 1-1 (λ_1) or section 2-2 (λ_2) can be approximated based on the load paths depicted in **Figure 3.5.19**. According to the force equilibrium condition in the node *E* or node *F* (**Figure 3.5.19**), the proportions needed to satisfy the following equation:

$$\lambda_1 = \frac{P_c}{P} = \frac{\tan \alpha_2}{\tan \alpha_1} \quad \text{or} \quad \lambda_2 = \frac{P_c}{P} = \frac{\tan \theta_2}{\tan \theta_1} \quad (3-4)$$

where α_1 , α_2 , θ_1 and θ_2 = Strut angles shown in **Figure 3.5.19**.

Figure 3.5.3 Analytical and Yanez's [Y2] experimental load-displacement responses

Figure 3.5.4 Analytical and Ali's [A4] experimental load-displacement responses

Figure 3.5.5 Analytical and experimental crack patterns

Figure 3.5.6 Principal stress flows of Specimen S1

Figure 3.5.7 Principal stress flows of Specimen S2

Figure 3.5.8 Principal stress flows of Specimen S3

Figure 3.5.9 Principal stress flows of Specimen S4

Figure 3.5.10 Principal stress flows of Specimen S5

Figure 3.5.11 Principal stress flows of Specimen S6

Figure 3.5.12 Principal stress flows of Specimen W-1

Figure 3.5.13 Principal stress flows of Specimen W-2

Figure 3.5.14 Principal stress flows of Specimen W-3

Figure 3.5.15 Principal stress flows of Specimen W-4

Figure 3.5.16 Zone division of squat wall with irregular openings

Figure 3.5.17 Zone division of slender wall with irregular openings

Figure 3.5.18 Zone division of squat wall with regular openings

(a) Positive direction

(b) Negative direction

Figure 3.5.19 Load paths for squat wall with irregular openings (with column contributions for transferring shear)

Figure 3.5.20 Load paths for squat wall with irregular openings (without column contributions for transferring shear)

Figure 3.5.21 Proportions of shear force carried by columns

Table 3.2 lists the comparison between the proportions (λ_1, λ_2) estimated by Equation (3-4) and the maximum proportions λ_{1Max} and λ_{2Max} obtained from the nonlinear finite element analysis of Specimens S2, S3 and S4. It can be seen that the proportion estimated by Equation (3-4) generally are less than the maximum proportions obtained from the nonlinear finite element analysis results. The proportions λ_1 and λ_2 of Specimen S2 or S3 only reach about 60% of λ_{1Max} and λ_{2Max} , whereas the proportions λ_1 and λ_2 of Specimen S4 reach about 90% of λ_{1Max} and λ_{2Max} .

Table 3.2 Maximum proportions of shear forces carried by columns

Specimens	α_1	α_2	θ_1	θ_2	λ_1	λ_{1Max}	λ_2	λ_{2Max}
S2	81.4°	54.1°	82.2°	44.2°	0.21	0.31	0.13	0.25
S3	75.4°	54.2°	76.1°	45.7°	0.36	0.51	0.25	0.45
S4	71.0°	58.2°	67.2°	47.3°	0.56	0.64	0.45	0.51

3.5.4 Effect of boundary elements

Normally the walls have boundary elements, such as flanges or barbell at the two ends. These boundary elements can significantly increase the flexural capacity of walls but at the same time excessive shear force may be imposed to the web. In order to investigate the influence of boundary elements, a $200 \times 400 \text{ mm}$ boundary element was added to each side of Yanez's specimens with irregular openings (S2 ~ S4) [Y2] as shown in **Figure 3.5.22**. Two vertical reinforcement ratios of $\rho_v = 0.5\%$ and $\rho_v = 0.8\%$ were adopted in the flanges. The loading history applied to each wall with boundary elements was the same as that of corresponding rectangular wall.

Figure 3.5.22 Specimens with boundary elements (200×400 mm)

Three groups of backbone envelopes of the load-displacement response loops are shown in **Figure 3.5.23**, including the envelopes of specimens S2, S3, and S4 with boundary elements of vertical reinforcement ratios of $\rho_v = 0.5\%$ and $\rho_v = 0.8\%$ respectively and the envelopes of rectangular specimens obtained from Yanez's experimental results [Y2]. **Figure 3.5.23** shows that the presence of boundary elements increased the lateral load resistance capacity significantly. When the reinforcement ratio in the boundary element was adequate, such as $\rho_v = 0.5\%$, the bar-bell shaped walls with irregular openings could achieve good ductility. However a slight increase in the reinforcement ratio in the boundary element to $\rho_v = 0.8\%$ could dramatically increase the lateral force and changed the failure mode from the compression of concrete in the boundary element to the tension failure in the beam zones which meant shear failure became critical. In order to obtain the necessary ductility level as shown in **Figure 3.5.23**, the reinforcement in the beam zone was increased from 6 high strength deformed bars with a diameter of 8 mm (6HD8) to 6 bars with a diameter of 10 mm (6HD10). Like the observation in Yanez's experiment [Y2], the size and arrangement of the openings did not have a significant effect on the behaviour of the bar-bell shaped walls with irregular openings. As illustrated by **Figure 3.5.23** the three specimens (S2, S3, and S4) displayed the similar strength capacities and ductility levels.

Figure 3.5.24 illustrates that the stiffness of Specimen S3 increases with the presence of the boundary elements. The higher vertical reinforcement ratio in the boundary element leads to the higher stiffness after top drift ratio 0.2%. Here the stiffness is the secant stiffness at the peak displacement of each cycle. Similar behaviour can be observed in the other two specimens. The size and arrangement of openings slightly influence the stiffness of walls with or without boundary elements. The stiffness curves of three specimens with boundary elements of $\rho_v = 0.5\%$ are plotted in **Figure 3.5.25**. They are very close to each other.

Figure 3.5.23 Backbone envelopes of load-displacement response loops of Specimens S2, S3 and S4 with boundary elements

Figure 3.5.24 Stiffness of Specimen S3 with and without boundary elements

Figure 3.5.25 Stiffness of specimens S2, S3 and S4 with flanges of $\rho_v = 0.5\%$

3.1 Summary

The main aspects in performing 2D and 3D nonlinear finite element analyses, including the element types, the material constitutive laws and the iteration methods for solution etc, are reviewed in this chapter. The 2D modelling method using WCOMD and the 3D modelling method using DIANA8.1 are introduced. The modelling techniques developed in this chapter will be applied in the following parts of the thesis when nonlinear finite element analysis is required. A comparison between the two programs on modelling reinforced concrete walls are performed in **Table 3.3**.

The 2D nonlinear finite element analysis results for Yanez's specimens [Y2] and Ali's specimens [A4] have been presented and discussed. It is noted that the 2D analysis using programme WCOMD can accurately simulate the experimental behaviour. The 3D analysis results will be presented in Chapter 6. In the 3D analysis, the reinforcement is modelled separately as truss elements with Von Mises yield criterion and work hardening material property. The total strain rotating crack model with parabolic compressive hardening-softening and exponential tension softening response is chosen to depict the behaviour of concrete.

A wall with irregular openings can be divided into beam zones, column zones, panel zones and nodal zones based on the distributions of its principal stresses. High tensile stresses generally occur in the beam zones or column zones. Diagonal principal

compressive flows are often observed in panel zones. Similarly a wall with regular openings can be divided into panel zones and beam zones.

Table 3.3 A comparison between Programs WCOMD and DIANA8.1

Programs		WCOMD	DIANA8.1
Dimension		2D	3D
Modelling	Concrete	Eight-node quadrilateral isoperimetric plane stress elements; Total strain fixed angle smeared crack model with tension softening and stiffening models developed at the University of Tokoy	Twenty-node isoperimetric solid brick element; Total strain rotating angle smeared crack model with parabolic compressive hardening-softening and exponential tension softening response
	Reinforcement	Embedded reinforcement; the constitutive law modified by bond effect	Separate truss element; normal constitutive law for steel bar
	Discontinuities	Joint element	Interface elements
Iteration method		Regular Newton-Raphson method	Regular Newton-Raphson method
Output	Stress	Principal stress trajectory and tables etc	Principal stress contours and tables etc
	Strain	Tables and figures etc	Contour and table etc

The load paths of a structure can serve as the basis for developing strut-and-tie models. It can be shown by the principal stress flows obtained from the nonlinear finite element analysis. The load paths of squat walls with irregular openings display good correlation to the strut-and-tie models applied by previous researchers.

Boundary elements can significantly influence the behaviour of a wall with irregular openings. If a proper vertical reinforcement ratio is chosen for the boundary elements, the walls can achieve higher strength, stiffness and ductility than rectangular walls.

CHAPTER FOUR

EXPERIMENTAL PROGRAMME OF FLANGED WALLS WITH OPENINGS

4.1 Introduction

The experimental study reported in this chapter was conducted to investigate the influence of flanges on the behaviour of a wall with openings. According to the results of the nonlinear finite element analysis performed in Chapter 3, the boundary elements can exert significant effect on the behaviour of walls, such as influencing the strength, stiffness, and ductility *etc.* The presence of the boundary elements considerably increases the moment resistance capacity of a wall but simultaneously jeopardising the shear resistance capacity compared to a rectangular wall. Piercing the wall with openings may further weaken its shear resistance capacity. However, it could be that the load path in the web of a wall with openings and boundary elements such as flanges may become clearer than in a rectangular wall with openings due to a higher induced shear force. Therefore, an experimental programme to investigate the behaviour of flanged walls with openings was carried out.

In order to compare the experimental results of Yanez [Y1, Y2], the configurations of specimens in this study were made identical to his specimens except that 120 mm thick by 400 mm deep flanges were added to the side edge of the wall to assess the effect of the flanges. The vertical reinforcement ratios in the web were chosen as close as possible to his specimens, while the reinforcement ratios in the beam and column zones of walls with irregular openings were slightly increased to consider the increased shear force induced by the presence of flanges based on the strut-and-tie models. No axial loading was applied in this test because the lateral force transfer mechanisms in the walls were the focus in this study.

4.2 Description of the Wall Specimens

Figures 4.2.1 to 4.2.6 depict the details of Specimens S-F1 to S-F6, respectively. The details of top beams and base beams of walls are presented in Figure 4.2.7. Like Yanez's specimens [Y2], each wall specimen in this test was 2000 mm wide, 2300 mm high and

120 mm thick, with aspect ratio $h_w/l_w = 1.3$, in which the height $h_w = 2540$ mm was the vertical distance between the lateral loading point and the wall base. The wall specimens consisted of one solid wall Specimen S-F1, three walls with irregular openings namely Specimens S-F2, S-F3 and S-F4, and two walls with regular openings namely Specimens S-F5 and S-F6. Specimen S-F1 served as the control specimen. Specimens S-F2, S-F3 and S-F5 had relatively larger openings of size 600 mm×600 mm compared with Specimens S-F4 and S-F6, which had 400 mm×400 mm openings. The openings of Specimen S-F2 were closer to the boundaries than those of Specimen S-F3.

High yield strength steel deformed bars and mild steel plain bars were used in the specimens. According to the BS4449 code [B5], the nominal yield strength of a high yield steel bar is $f_y = 460$ N/mm² and the nominal yield strength of a mild steel bar is $f_y = 250$ N/mm². The high yield strength steel deformed bars and the mild steel plain bars are indicated as T bars and R bars respectively. The reinforcements applied in a certain place are denoted according to their quantity, steel types and diameters. For example, 8T10 bars means there are eight T bars whose diameters are 10 mm. The concrete applied in this test was Grade 30 with a characteristic strength $f'_{cu} = 30$ N/mm² as specified in the BS 8110 code [B6], which is determined according to the compressive strengths of 150×150×150 mm concrete cubes at 28 days.

Similar to Yanez's design procedure, the flanged solid wall Specimen S-F1 was first designed using the conventional design method for a flexural member. The web horizontal and vertical reinforcement ratios were determined as 0.5%, which were slightly larger than Yanez's solid wall Specimen S1. High yield strength steel bars were applied in the web as main bars, which were the reinforcing bars applied to take outer forces. However, only mild steel bars of lower nominal yield strength were applied in the flanges with consideration to the limit of the tensile capacity of the testing rig.

The vertical reinforcements of the flanges of other five specimens were kept the same as the flanges of Specimen S-F1 of (8R10). The web's main reinforcements of flanged walls with irregular openings (Specimens S-F2 to S-F4) were arranged to the zones where the ties were located according to the strut-and-tie models presented in **Figures 4.4.1 to 4.4.3**, but at the same time the numbers of vertical and horizontal reinforcements were also maintained as close as possible to those of Specimen S-F1. In the other zones, secondary

reinforcements, which were assigned to limit the crack opening, were applied. Therefore, the vertical main bars of the flanged walls with irregular openings were distributed at the column zones between the openings and the bottom panels, while the horizontal main bars were distributed in the beam zones. High yield steel deformed bars were applied as the main bars and the mild steel plain bars were used as secondary reinforcements.

The flanged walls with regular openings (Specimen S-F5 and S-F6) were designed using an elastic analysis method of frames with rigid ends in the beams as shown in **Figures 4.4.4 and 4.4.5**. In the webs of these two walls, the total number of the vertical bars was the same as that for Specimen S-F1. They were distributed to the two parts of the web equally. The numbers of horizontal deformed bars in the two beam zones were the same as those of the walls with irregular openings. Secondary reinforcement was applied in the other zones.

4.3 Material Properties

4.3.1 Concrete

Two batches of the ready-mix concrete, which had a characteristic strength of $f'_{cu} = 30 \text{ N/mm}^2$, 13 mm maximum size aggregate and a slump 100 mm, were used to cast the specimens. In total twenty-four $150 \times 150 \times 150 \text{ mm}$ cubes and $150 \times 300 \text{ mm}$ cylinders were cast and cured following to the standard procedures. The cylinder compressive strengths f'_c and cube compressive strengths f'_{cu} were tested on the 28th day after they were cast and on the day of each specimen test. Three cubes and cylinders were tested to determine the corresponding average compressive strengths, which are listed in **Table 4.1**.

4.3.2 Reinforcement

Table 4.2 presents the tested properties of the reinforcing bars, which were the averages of the tested properties of three steel bar specimens. The yield strengths and strains were the corresponding values obtained at the yield points because both the plain mild steel bars and high yield steel bars showed clear yield points though the yield plateau of plain mild steel bar were longer than those of high yield strength deformed bars as indicated in **Figure 4.3.1**. The ultimate strengths were the maximum strength recorded in the test. T10, R10 and R6 bars were used in the walls while T13 and T20 bars were applied in the loading beams and base beams.

- Notes:
1. Nominal yield strengths of R bars and T bars are 250MPa and 460MPa respectively.
 2. Except for the vertical bars of 8R10 in each flange, the vertical and horizontal bars in the web are T10.
 3. The stirrup arrangement in the flange is R6@200.

Figure 4.2.1 Details of Specimen S-F1

- Notes:
1. Except for the vertical bars in each flange (8R10) and the R10 bars specified in the figure, other vertical bars are T10.
 2. Except for T10 bars specified in the figure, other horizontal bars are R10.
 3. Except for the stirrup arrangements specified in the figure (R6@200), the arrangements in other regions of flanges are R6@200.

Figure 4.2.2 Details of Specimen S-F2

Figure 4.2.5 Details of Specimen S-F5

Figure 4.2.6 Details of Specimen S-F6

Figure 4.2.7 Details of top and base beams

Table 4.1 Measured compressive strength of concrete

Specimens	Age at test day (days)	¹ Average f'_{cu} (N/mm^2)	² Average f'_c (N/mm^2)
S-F1 ~ S-F4	28	42.88	34.95
S-F5 ~ S-F6	28	41.77	33.83
S-F1	33	44.74	35.02
S-F2	39	46.66	36.86
S-F3	48	43.80	35.05
S-F4	44	44.47	36.26
S-F5	55	43.80	35.84
S-F6	51	43.18	34.98

1. f'_{cu} concrete cube compressive strength

2. f'_c Concrete cylinder compressive strength

Table 4.2 Measured steel bar properties

Types	Yield strength f_y (N/mm^2)	Yield Strain ϵ_y	Ultimate strength f_u (N/mm^2)
R6	293	0.002030	405
R10	382	0.001913	481
T10	467	0.002612	541
T13	493	0.002839	581
T20	512	0.003495	607

Figure 4.3.1 Stress-strain relationships for steel reinforcements

4.4 Pre-test Analysis of Six Flanged Wall Specimens

This section demonstrates the details of the design and the pre-test analysis of six flanged wall specimens. The ideal strengths (P_i) of six specimens presented here were computed based on the tested material strengths rather than on their nominal strengths.

The ideal strength of solid flanged wall Specimen S-F1 was determined by the ideal moment capacity of its base section. It could be accurately evaluated according to common methods for the design of a flexural member introduced by many textbooks [M1, P3]. Using procedure from NZS 3101[N6], the ideal strength was then calculated as $P_i = \pm 353.3 \text{ kN}$, and the shear capacity provided by the concrete and horizontal bars was 718.7 kN , which was much higher than $P_i = \pm 353.3 \text{ kN}$.

The ideal strengths of flanged wall specimens with irregular openings (S-F2 ~ S-F4) were estimated by strut-and-tie models presented in **Figures 4.4.1 to 4.4.3**. In these models, the flanges were assumed as axially tensile or compressive components. The horizontal ties were located in the middle of the beam zones. **Figure 4.4.1** depicts the strut-and-tie models of Specimen S-F2 for the negative and positive loadings. Based on the models, the negative ideal strength was determined as $P_i = -214.2 \text{ kN}$ by the vertical tie **AG**, where 6T10 bars were arranged. Controlled by the capacity of the horizontal tie **LK**, the positive ideal strength of wall reached $P_i = 220.0 \text{ kN}$. In the model for positive loading, the position of node **L** was decided by the resultant of forces provided by the steel bars of 8T10. Assuming that the effective compressive strength of concrete was $f'_{ce} = 0.68 f'_c$, as suggested by Schlaich *et al* [S2, S3], for compression field with skew cracks, the minimum requirement width of strut **LQ** was determined, and in turn the position of node **Q** was located. Presuming that the compressive force of strut **LQ** sprays out by an angle 25° from the ends, which was the minimum angle between a strut and a tie as stated by ACI 318 [A1], the secondary reinforcement in the panel zone was designed according to the tensile force **T** as demonstrated in **Figure 4.4.1**.

Figure 4.4.1 Possible strut-and-tie models of Specimen S-F2

Figure 4.4.2 Possible strut-and-tie models of Specimen S-F3

Figure 4.4.3 Possible strut-and-tie model of Specimen S-F4

The strut-and-tie model of Specimen S-F3 for negative loading was identical to that of Specimen S-F2. The same negative ideal strength of $P_i = -214.2 \text{ kN}$ was achieved. However, differences were apparent between the models of these two specimens for positive loading. According to the model for Specimen S-F3, all of the vertical bars in the middle column zones were possible under compression in the bottoms, so the tie **LN** became most critical in all components. Considering that the vertical bars (tie **SM**) in the flange generally yielded earlier than bars in the bottom panel, the strut **SQ** could sustain a part of the shear force as indicated by **Figure 4.4.2**. The positive ideal strength then was determined as $P_i = 203.3 \text{ kN}$.

The strut-and-tie models of Specimen S-F4 were developed in a similar way for Specimen S-F3 as plotted in **Figure 4.4.3**. Its negative and positive ideal strengths were limited by the capacities of ties **BD** and **LK** respectively, they were the same as $P_i = \pm 220.0 \text{ kN}$. Considering the contribution of tie **SM** to transfer shear force, the tie **LK** became more critical than the vertical ties in the bottom of the model for positive loadings.

The ideal strengths of flanged wall Specimens S-F5 and S-F6 were decided based on the elastic analysis on frame models with rigid ends to the beams. **Figures 4.4.4** and **4.4.5** display the frame models and their bending moment diagrams. The ideal strength is reached when any wall section reaches its capacity, which generally occurs in the tension wall. In the analysis the equivalent moment of inertia (I_e) were taken as what suggested by Paulay [P7]. They were $I_e = 0.5I_g$ and $I_e = 0.8I_g$ for tension walls and compression walls respectively, where I_g is the moment of inertia of the uncracked gross concrete section, and the equivalent moments of inertia (I_e) for coupling beams in the walls were calculated by equation:

$$I_e = 0.2I_g / [1 + 3(h/l_n)] \quad (4-1)$$

where, h, l_n = depth and clear span of coupling beam

The effective moments of inertia of top beams were taken as $I_e = 0.5I_g$ because there were normally no cracks opening on them during test. The length of the rigid end of beam was determined as the distance of the centroid of each wall to inner boundary subtracted to $h/4$, where h was the depth of the coupling beam. The ideal strengths were then determined as $P_i = \pm 318 \text{ kN}$ and $P_i = \pm 321.8 \text{ kN}$ for Specimens S-F5 and S-F6,

respectively. The shear capacities of the coupling beams of Specimens S-F5 and S-F6 were 49.7 kN and 89.3 kN , higher than those required by Figures 4.4.4 and 4.4.5.

Figure 4.4.4 Moment diagram of Specimen S-F5 frame model (kNm)

Figure 4.4.5 Moment diagram of Specimen S-F6 frame model (kNm)

4.5 Construction of Wall Units

The six wall specimens were horizontally cast in the workshop of the contractor's company. **Figure 4.5.1** shows the photographs of the formworks and steel cages of all specimens. The formworks, made from plywood sheets and stiffened by timber blocks, were designed and inspected according to BS 8110 [B6]. Horizontal levels of wall surfaces were especially checked to ensure that no eccentricities occur. The surfaces of the moulds were oiled before the concrete casting and special clampers were applied to ensure the positions of preset components and the holes.

The reinforcing bars were cut and bent according to their design length, within a tolerance $= \pm 2\text{ mm}$. The reinforcement cages were constructed by the following procedures. First, the reinforcement grids of the webs and the flanges were fabricated separately. After the strain gauges in the bars were attached to their surfaces and the wires of gauges were arranged, the grids and the flange were set up together to form the steel cages. At last, the cages were put into the formwork and the positions of reinforcement bars were adjusted to satisfy the tolerance.

The specimens were cast in two batches. Specimens S-F1 to S-F4 were cast firstly when the formworks of Specimens S-F5 and S-F6 were still being built. The other two specimens were cast three days later. The concretes of all specimens were compacted using an internal vibrator. Special attention was paid to the positions of joints, such as the joints between the base beams and the walls as well as those between the flanges and the webs, to ensure good concrete quality there. The outer surface of every specimen was made extra smooth for drawing crack patterns during test. After casting, the walls were covered with wet Hessian and nylon to keep damp for a week. The concrete cubes and cylinders were stored near the walls and cured under the same condition.

4.6 Loading Frame and Loading Sequences

4.6.1 Testing rig

The testing rig, shown in **Figure 4.6.1**, consisted of two systems: an in-plane loading system and an in-plane base beam reaction system.

The in-plane loading system included a double action hydraulic jack and four steel beams. The jack had a capacity of 2000 *kN* in compression and 1200 *kN* in tension. The stroke of the jack was 405 *mm*. The steel frame was arranged in such a configuration so that the

two direction lateral loadings could be applied to the wall. Four high strength steel rods were preset in the loading beam to enable the lateral loading to be introduced as compression on the end faces of the top loading beam.

The base beam reaction system was designed to resist the rotation and sliding of the wall specimen when the load was applied. They were attached to a strong floor by high strength rods. Reaction footings were provided to balance the lateral loading. A prestress scheme was applied to every steel rod so that the rotation and sliding during the test could be restrained efficiently.

4.6.2 Loading sequence

All specimens were tested to failure under quasi-static loading sequences. No axial load was applied. The reversed cyclic lateral loading was applied to the top of the wall according to the loading sequence recommended by Park [P2] as presented in **Figure 4.6.2**. The loading sequence began with three load-controlled cycles, during which it was assumed that the structure was still within the elastic state. The horizontal loadings increased from $\pm 0.25P_i$ to $\pm 0.75P_i$ by $\pm 0.5P_i$, where P_i was the ideal strength of the test specimen. In Specimen S-F1, it was associated with the nominal flexure strength M_i being reached at the bottom section. Following the load-controlled cycles, the displacement-controlled cycles started. The displacement was increased according to the ductility factor (DF or μ_Δ). It was calculated by equation

$$DF = \Delta / \Delta_y \quad (4-2)$$

where Δ = top drift imposed to the structure

Δ_y = structural yield displacement

Park also suggested a scheme to define the yield displacement according to the top drifts $\Delta_{0.75}$ obtained from $\pm 0.75P_i$ cycles as shown in **Figure 4.6.3**. The yield displacement of the structure Δ_y was calculated by the following equation according to the scheme.

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

In ductility factor $DF = \pm 1$, only one load cycle was applied. After that, two load cycles were applied in each successive ductility level, which was $DF = \pm 2, \pm 3, \pm 4, \dots$. The end of the test was reached when the strength dropped to less than 80% of the recorded maximum loading.

(a) Specimen S-F1

(b) Specimen S-F2

(c) Specimen S-F3

(d) Specimen S-F4

(e) Specimen S-F5

(f) Specimen S-F6

Figure 4.5.1 Moulds and steel cages of specimens

Figure 4.6.1 (a) Details in drawing

Figure 4.6.1 (b) Details in photograph

Figure 4.6.1 Details of testing-rig

Figure 4.6.2 Imposed displacement history used to establish the available ductility factor, from Park [P2]

Figure 4.6.3 Definition of yield displacement, from Park [P2]

4.7 Instrumentation

The objectives of the instrumentation on the wall specimens were to observe the following parameters:

- 1) lateral force imposed by the hydraulic jack,
- 2) lateral deflection at the top of the wall,
- 3) flexural deformation in the wall unit,
- 4) shear deformation in the wall unit,
- 5) horizontal sliding of the wall unit, and
- 6) strains in selected reinforcing bars.

The lateral force was measured by the load cell connected to the jack as displayed in **Figure 4.6.1**. The parameters (2) to (5) were detected by the linear variable differential transducers (LVDT) mounted on the specimens. The strains of reinforcing bars were measured using electric resistance wire strain gauges attached on them.

4.7.1 Load Cell

In order to measure the applied load, a double-acting (compression and tension) load cell with capacity 2000 kN was connected to the hydraulic jack using high yield strength steel rod with threads as presented in **Figure 4.6.1**. The lateral loading applied to the top of the wall was 0.67 times of the measured load.

4.7.2 Strain gauges

FLA type 5 mm-gauge length strain gauges, which has 10 m vinyl-insulated lead wires, were used to measure the local strains in the steel reinforcing bars. In order to ensure that the local strains of the reinforcements in the critical zones, such as the bottom of the wall, beam zone or the column zone were measured, two strain gauges were arranged at the same position on both reinforcing bar grids. A staggered distribution scheme of strain gauges was applied in the other zone, where the strain gauges were alternatively distributed between two steel bar grids. **Figures 4.7.1 to 4.7.6** presents the details of strain gauge distributions of all specimens.

The strain gauges were represented by two numbering systems. First system consisted of unique item number for each strain gauge, which was marked beside the strain gauge as plotted in **Figures 4.7.1 to 4.7.6**. The number of second system contained the message of

gauge positions, including the bar position on which the gauge was mounted and the gauge position on the bar. It consisted of a code for the reinforcing bar grid, a code for the direction of reinforcing bar, a bar number and a position code. As shown in the figures, these codes were marked near the details of the reinforcement layouts of the specimens. The codes of reinforcing bar grids consisted of **S** and **N** representing the reinforcing bar grids in the web, and **A**, **B**, **C**, and **D** representing the bar grids in the flanges. **H** and **V** were the codes of bar directions. **H** referred to a horizontal bar while **V** indicated a vertical bar. Position codes were a serial of number representing the position in different vertical levels or horizontal positions. For example, the gauge of number SV1-1 of Specimen S-F1 means the gauge was mounted in bar grid **S**, the bar where it was mounted was a vertical bar with bar number 1 and the gauge was located at the vertical position 1. Referring to **Figure 4.7.1**, it was number #1 gauge in the first number system.

4.7.3 Linear variable differential transducer (LVDT)

Four types of Linear Variable Differential Transducers (LVDT), with 300 mm travel, 100 mm travel, 50 mm travel and 25 mm travel, were applied to measure the top drifts, the flexural deformations and the shear deformations of the wall specimens. **Figures 4.7.7 to 4.7.12** present the distribution of transducers in six specimens. LVDT with 300 mm travel, numbered as #L1, was set up at the centre of the face of the top beam end to measure the top drift of each wall specimen. LVDTs arranged vertically in the walls were applied to detect the flexural deformation. The panel shear deformations were measured by the LVDTs distributed along diagonal directions of the panels. Two LVDTs were horizontally installed at the positions slightly above the wall base to record the sliding of each specimen. The rotation and sliding of base beam were measured by three LVDTs with the last three numbers as displayed in the figures.

4.8 Evaluation of Wall Deformations

4.8.1 Wall deformations

Four types of deformation, including the shear distortion, the flexural distortion, and the fixed-end rotation about the centre of the compression zone in a wall base as well as sliding, may occur in a solid wall as demonstrated in **Figure 4.8.1**. After a wall crack, the shear distortion comes from the sliding and opening at the diagonal cracks. The fixed-end rotation of the wall may be caused by straining plus bond-slip occurring on the vertical

reinforcing bar below the wall base. **Figure 4.8.1** also depicts the LVDT distribution scheme to measure these deformations. All four types of deformations make contributions to the top drift of wall in the way presented in the figure. They were Δ_v caused by the shear, Δ_f evoked by the flexure, Δ_R induced by the fixed-end rotation and Δ_s resulted from sliding.

4.8.2 Evaluation of shear deformation of wall

The shear distortion of a rectangular panel can be evaluated based on the length variations of its two diagonals as shown in **Figure 4.8.2**. The deformation is given by the equation

$$\gamma = \gamma_1 + \gamma_2 = \frac{\Delta_2 - \Delta_1}{2L} (\tan \alpha + \cot \alpha) \quad (4-3)$$

where Δ_1, Δ_2 = deformations of the panel diagonals, which are positive when they are in extension

L = length of the diagonal

α = angle between the diagonal and the side of rectangle

The top drift due to shear distortion Δ_v can be calculated by

$$\Delta_v = \gamma h_w \quad (4-4)$$

where h_w = distance from the base of the wall up to lateral loading point as displayed in **Figure 4.8.1 (a)**

Hiraishi [H1] and Massone and Wallace [M11] have pointed out that using Equation (4-3) to determine the shear deformation of a wall is valid only when the curvature of the wall is uniformly distributed, while in other cases the shear deformation calculated from Equation (4-3) should be corrected according to its flexural deformation. Thus the accuracy of the measurement of the shear deformation is dependent on the measurement of the flexural deformation. Because no new reliable and independent method for the measurement of the shear deformation has been suggested, the Equation (4-3) is still applied. However, the measured regions of the LVDTs for the measurements of Δ_1 and Δ_2 were chosen 50 mm above the wall base, so that the base of the wall where the flexural distortion concentrated could be avoided.

4.8.3 Evaluation of flexural deformation of wall

Based on the Bernouli hypothesis that plane sections remain plane after deformation, the wall flexural deformations can be obtained from the rotation of each segment in the wall measured from a pair of left and right LVDTs located at the face of two flanges. With reference to **Figure 4.8.3**, the change of slope of region i , is given by

$$\theta_i = \frac{\delta_{Ri} - \delta_{Li}}{L_T} \quad (4-4)$$

where θ_i is the change of slope occurring in the region i , δ_{Ri} and δ_{Li} are the extension and shortening of right and left flange measured by a pair of LVDTs in the region i , and L_T is the horizontal distance between left and right LVDTs. The average curvature ϕ_i in the region i can be evaluated as

$$\phi_i = \frac{\theta_i}{S_i} \quad (4-5)$$

where S_i is the gauge length of LVDT in this region. The top drift obtained from the change of slope in this region is

$$\Delta_{fi} = \theta_i (h_w - h_i) = \theta_i H_i \quad (4-6)$$

where h_i is the vertical distance between the centre line of region i to the wall base and H_i is the vertical distance from the centre line to the loading point. Thus, the top drift induced by flexural distortion Δ_f can be obtained from the sum of Δ_{fi} based on this discrete model. As shown in **Figure 4.8.4**, assuming that three pairs of LVDTs are used to measure the flexural distortions of regions 1, 2 and 3, the top drift caused by flexure then can be obtained by

$$\Delta_f = \Delta_{f1} + \Delta_{f2} + \Delta_{f3} \quad (4-7)$$

It should be noted that the slope change in region 1, just adjunct to the base joint, may include the distortion due to the joint open. The slope change induced by the fixed-end rotation as depicted in **Figure 4.8.1 (c)** then is averaged into the slope change of region 1. Therefore the curvature of this region is possible to be overestimated by Equation 4-5.

Figure 4.7.2 Strain gauge positions of Specimen S-F2

Figure 4.7.4 Strain gauge positions of Specimen S-F4

Figure 4.7.6 Strain gauge positions of Specimen S-F6

Figure 4.7.7 Positions of LVDTs in Specimen S-F1

Figure 4.7.8 Positions of LVDTs in Specimen S-F2

Figure 4.7.9 Positions of LVDTs in Specimen S-F3

Figure 4.7.10 Positions of LVDTs in Specimen S-F4

Figure 4.7.11 Positions of LVDTs in Specimen S-F5

Figure 4.7.12 Positions of LVDTs in Specimen S-F6

Figure 4.8.1 Wall deformations

Figure 4.8.2 Shear distortion in a rectangular panel

Figure 4.8.3 Distribution of LVDTs for the evaluation of the wall flexure deformation

Figure 4.8.4 Relationship of the flexural distortion and top drift

CHAPTER FIVE

PRESENTATION OF TEST RESULTS

5.1 Introduction

In this research programme, it is assumed that the test specimen lies in the plane of the paper and it is viewed from the south. The positive loading and lateral displacement are toward the right (east) so that in the experiment, the hydraulic jack was set up on the east (right-hand side) of the test specimen. The lateral loading in this test began in the negative direction (west), and all photographs, diagrams and testing results in this chapter were presented according to this assumption.

This chapter presents the test results of six approximately 1/3-scale model flanged wall specimens. The characteristic behaviour of each specimen is presented according to the loading stages separately. The description of the experimental responses includes the measured specimen strengths and deformations as well as the strain measurements for the reinforcing bars. The main properties of tested specimens are summarised in Table 5.1, which is presented in Section 5.8. Particular loading stages in a test are expressed as displacement ductility factor (DF), for example, $DF = \pm 1(2)$ designates the second positive or negative cycle of $DF = 1$. Displacement ductility factor (DF) is often also expressed as μ_{Δ} by some researchers [G1, P7]. In this test, ductility factors (DF) of specimens were determined to be slightly higher than their actual ductility level because they were based on a relatively lower yield displacement obtained from the test of Specimen S-F1 using the method stated in Section 4.6.2. When compared with other experimental results or code specifications, the displacement ductility factors of Specimens (DF) obtained from the test will be converted into their actual displacement ductility factor (μ_{Δ}) according to the experimental load-displacement response loops and the strain statuses of their reinforcing bars, which will be expressed as μ_{Δ} , in the following chapters. As ductility factor DF of a specimen deviated from its actual value, in this chapter top drift ratio δ was also presented as another index to reflect the deformation abilities of walls. A detailed discussion on the test results will be presented in Chapter 6.

Before presenting the test results, all records of the LVDTs installed to measure the sliding and rotation of the base beams of walls were checked. It was found that the

maximum record of all LVDTs was less than 0.15 mm, which was much less than the deformations in walls, so the influence of deformations in the base beams to the wall behaviour could be ignored.

5.2 Test Results of Specimen S-F1

Specimen S-F1, as shown in Figure 4.2.1, was a solid wall serving as a control specimen in this experiment. The yield displacement of this specimen determined using the method stated in Section 4.6.2 would also be applied to successive specimens for the determination of their loading histories. Test results indicated that this yield displacement was lower than its actual value. A detailed discussion of the evaluations of the yield displacement and ductility of a specimen will be presented in Section 6.3.

5.2.1 Loading history

Figure 5.2.1 shows the loading history applied to the Specimen S-F1. Every dot indicated the top displacement introduced to the specimen. The first three cycles were load-controlled cycles, followed by the displacement cycles. $0.25P_i$, $0.5P_i$ and $0.75P_i$ were applied in load-controlled cycles. In the test, $P_i = 286\text{ kN}$ calculated according to the code specified nominal strengths of steels was applied to determine the yield displacement Δ_y . It was 3.2 mm based on the top drifts obtained in $\pm 0.75P_i$ cycles. The loading was ended at $DF = 10$ when the strength of specimen was less than 80% of the recorded maximum strength and its failure mechanism was clearly observed.

Figure 5.2.1 Loading history of Specimen S-F1

The ideal strength of wall $P_i = 286 \text{ kN}$ was lower than expected because it was calculated according to the code specified nominal strengths of steels. As indicated in BS4449 [B5], the nominal yield strength of a high yield strength bar is $f_y = 460 \text{ N/mm}^2$ and the nominal yield strength of a mild steel bar is $f_y = 250 \text{ N/mm}^2$. However, the measured yield strength of the mild steel bar was $f_y = 382 \text{ N/mm}^2$, which is much higher than its nominal yield strength. This caused an underestimation on the actual strengths of the specimens and in turn caused the ductility levels of the specimens to be overestimated. If calculated according to the measured strength of material, the ideal strength of the wall P_i reached 353.3 kN as stated in Chapter 4, which was close to the test result.

5.2.2 Description of observed behaviour

The behaviour of the specimen is outlined according to the loading stages below.

Loading stage	Description
$0.25P_i$	The specimen was in the elastic range. No cracks were observed.
$0.5P_i$	Flexural cracks appear in the bottom of the flanges. Two of the cracks extend into to the web, sloping down to the wall base. They penetrated into the web by about 500 mm .
$0.75P_i$	Figure 5.2.2 (a) displays that more flexural cracks formed in lower parts of the flanges and one of them extended into to the web as an inclined crack and propagated to the wall base during the negative or positive loading cycle. In this and the following crack pattern figures, the dashed lines refer to the cracks opened during the negative (west) loading cycle (negative crack), while the continuous lines depict the cracks opened during the positive (east) loading (positive crack).
$DF = \pm 1$	New flexural cracks continued opening in the flanges. One of the positive cracks and negative cracks extended into the web and inclined to the wall base as shown in Figure 5.2.2 (b) . An obvious opening was observed at the joint face of the wall and base beam. The width of the crack was 0.4 mm .

- $DF = \pm 2$ Most of the flexural cracks in the flanges and inclined cracks in the web formed in the loading cycles at this ductility. With a thudding noise, three long main inclined cracks developed from the upper half of the right flange and extended toward the wall base at an angle of about 35° , including ① cracks as presented in **Figure 5.2.2 (c)**. During the positive loading cycle, two long inclined cracks, including the ② crack, also appeared and extended through the web to the bottom of the wall. Generally no new cracks were formed during the second loading cycle of the same ductility level. The crack patterns opening under the negative and positive loadings were asymmetric, with the negative cracks being a little flatter than the positive cracks. The spacing between the negative cracks was less than the one between the positive cracks. The existing cracks kept on opening and extending. The width of cracks at the joint reached 0.8 mm .
- $DF = \pm 3$ The maximum positive lateral loading was reached in the first cycle at $DF = 3$. No more cracks were formed during these cycles, but the cracks
~
 $DF = \pm 4$ continue opening and extending. The horizontal cracks opening in the joint between the wall and base beam penetrated to the whole section at $DF = \pm 4$, which means that sliding face formed along the joint. Deformations began to concentrate at the cracks along the joint with the width of the cracks reaching 5 mm . The inclined crack opening during the cycle of $0.5P_i$ reached 0.8 mm . Because the crack pattern of the specimen was almost formed at $DF = \pm 4$, the ductility factor increased by 2 in the following cycles.
- $DF = \pm 6$ Deformation was further concentrated into the joint between the wall and the base beam. Horizontal cracks at the bottom of the flanges opened to more than 10 mm and the maximum negative lateral loading was reached in this ductility. Because the high compression occurred at the bottom of the flanges, the vertical split cracks were observed there. The concrete in the bottom of the east flange began to spall.
- $DF = \pm 8$ Vertical splitting and spalling of the concrete of the east flange occurred within the region about 200 mm above the joint, whereas only the concrete below the sliding face of the west flange was crushed. The spoiled region was

much smaller than that of the other side. The sliding of the wall could be visually observed, which induced the concrete near the joint to crush into fine grains. Most of the vertical bars in the flanges buckled at the wall base.

$DF = \pm 10$ The peak positive lateral loading of the first cycle in this ductility was less than the 80% of the maximum strength obtained in previous cycles, which meant that failure occurred. **Figure 5.2.3** illustrates the concrete in the bottom of the east flange was severely crushed and spalled. Some vertical bars in the flanges fractured at the sliding face level as shown in **Figure 5.2.4**. **Figure 5.2.5** presents the sliding face and the crushing of concrete along it.

5.2.3 Crack pattern and failure mechanism

Figure 5.2.6 displays the crack pattern of Specimen S-F1. The blue lines represent the cracks formed during the negative loading cycles, while the red lines represent the cracks formed during the positive loading cycles. The cracks opened first in the flange as flexure cracks then penetrated into the web as the inclined cracks. Most of the inclined cracks extended across the entire web, reaching the base of the wall. Clear diagonal struts could be observed in this crack pattern. The slopes of struts formed in the negative loading (negative struts) were about 35° , while the slopes of the positive struts (formed during the positive loading) were about 40° . No inclined cracks were observed in the region near the top beam due to its comparatively high level of stiffness.

A typical sliding failure, which often occurred in a wall with low aspect ratio [P7, P8, S1], was observed as illustrated in **Figure 5.2.7**. It is considered likely that failure was initiated by the interconnecting of horizontal cracks along the joint face after the significant yield of the vertical flexural bars. A horizontal shear path then formed, allowing the sliding displacement to occur. After the reinforcing bar in the compression zone yielded and buckled, significant sliding displacement occurred and the concrete was crushed into fine particulates along the sliding face. The sliding face, however, was not flat along the joint face. A bulge partly existed at the right side of the sliding face as displayed in **Figure 5.2.7**. This fact may influence the magnitude of the sliding displacement as it can be seen later that the sliding displacement in the negative loading direction was much larger than that occurring in the other direction. Under the positive loading, the rotation of the wall around the compression part of the bulge was significant. The effect existed until the concrete in the bulge was crushed under high compression.

The steel bars buckled and kinked under the action of sliding and compression and finally some of them fractured under tension when the load was reversed as shown in **Figures 5.2.3** and **5.2.4**. Because of the sliding, the stiffness, strength and the energy dissipation of the wall decreased significantly, plus deformations were concentrated in the sliding face, so there was not obvious slip and openings along the main diagonal cracks, such as crack ①, and ② depicted in **Figure 5.2.2**, were detected.

(a) At $\pm 0.75P_i$

(b) At $DF = \pm 1$ ($\delta = \pm 0.125\%$)

Figure 5.2.2 Crack development of Specimen S-F1 (cont. next Page)

(c) At $DF = \pm 2$ ($\delta = \pm 0.25\%$)

(d) At $DF = \pm 10$ ($\delta = \pm 1.25\%$)

(Cont.) Figure 5.2.2 Crack development of Specimen S-F1

Figure 5.2.3 Concrete crushing in the bottom of the east flange

Figure 5.2.4 Steel bar fracture in the west flange

Figure 5.2.5 Sliding face of Specimen S-F1

Figure 5.2.6 Crack pattern of Specimen S-F1

Figure 5.2.7 Failure mechanism of Specimen S-F1

5.2.4 Lateral load-top displacement relationship

Figure 5.2.8 presents the lateral load versus top displacement response loop of Specimen S-F1. It can be seen that the maximum negative and positive strengths were -393.3 kN and 371.9 kN , which exceed the ideal strengths $P_i = \pm 353.3 \text{ kN}$ by 6% and 12% of the positive and negative loadings respectively. After $DF = \pm 2$, large inelastic displacements were observed. Generally, the strength of the second loading cycle in a ductility level was less than that of the first loading cycle: the difference was about 10% in the positive loading, while it was 5% larger in the negative loading cycle after $DF = \pm 4$ because of the influence of sliding. The maximum negative strength was about 1.06 times of that of the positive one. A definite pinching was observed after the cycles of $DF = \pm 2$ and this

became significant as the loading increased because of the sliding. The angle of the hysteretic loop flattening means stiffness degradation occurred as the top drift increased. The stiffness at $DF = \pm 1$ was $K_1 = 75.4 \text{ kN/mm}$, which was the mean of the wall stiffness under the negative and positive loadings at the first cycle of $DF = \pm 1$.

The wall failed at a top drift ratio of 1.25% or $DF = 10$ as demonstrated in **Figure 5.2.8**. The ductility factor was comparatively higher than that of the common low walls [P7] because $\Delta_y = 3.2 \text{ mm}$ used to define it, is relative lower than it should be. The yield displacement was determined according to $P_i = 286 \text{ kN}$ calculated from the nominal strength of the mild steel bar. If $P_i = \pm 353.3 \text{ kN}$ calculated from the measured strength of the mild steel bar was applied, the yield displacement would be higher than that. Because this yield displacement was also applied to the left five specimens for the determination of their displacement ductility factors (DF), the actual ductility level of all specimens were also lower than those indicated by DF . Therefore the actual ductility levels of all specimens had to be again evaluated according to test results, which were presented in Section 6.3, and the actual ductility factor were designated as μ_Δ . Thus in this thesis, the ductility factor DF determined by $\Delta_y = 3.2 \text{ mm}$ was used only in the cases relative to this test. In other cases, such as the comparison with other experimental results, ductility factors μ_Δ determined in Section 6.3 using the actual yield displacement of structures according to the test results were applied. Because DF failed to represent the ductility levels of specimens, in this chapter the top drift ratios δ of walls were used as the indications of their deformation abilities. It could be achieved from DF through the equation.

$$\delta = 3.2 \times DF / h_w \times 100 \quad (5-1)$$

where h_w = wall height, taken as 2540 mm for all specimens

According to Equation (5-1), $DF = 8$ was corresponding to the top drift ratio $\delta = 1\%$ and when DF increased by 1, the drift ratio δ increased to 1.25%.

5.2.5 Components of top drift

As mentioned previously in Chapter 4, the top drift Δ consists of four parts: the flexural deflection part Δ_f , the shear deformation part Δ_v and the part induced by the fixed-end rotation Δ_R as well as the part caused by the sliding in the bottom Δ_s . The wall

deformation Δ_w presented in **Figure 5.2.7** consists of Δ_f and Δ_v , in which the flexural deflection Δ_f includes the part induced by the fixed-end rotation Δ_R .

Figure 5.2.8 Lateral load-top displacement response of Specimen S-F1

As shown in **Figure 4.7.7**, the flexural deflection Δ_f was measured by four groups of the LVDTs, including L2 & L6, L3 and L7, L4 & L8, and L5 & L9. The drift caused by the panel shear deformation Δ_v was measured by the L10 and L11. Sliding displacement had not been detected successfully by L13 because the sliding face occurred higher than its level or the range of LVDT was exceeded. In these cases the left part of sliding displacement were approximated by subtracting Δ_f , Δ_v and Δ_s recorded by the LVDT from the top drift Δ , and were regarded as unaccounted part in the figure.

Figure 5.2.9 illustrates the relative deformation portions of Specimen S-F1 varying with loading cycles. In the positive loading cycles, the sum of the flexural deflection Δ_f , shear deformation Δ_v and sliding Δ_s was close to the top drift. The largest error occurring in cycle $DF = 8(2)$ was within 10%. The flexural portion was higher than 70% and increased to more than 90% as the ductility level increased. It should be noted that this flexural drift includes the part caused by the fixed-end rotation. The details of the components of top drift of Specimen S-F1 were also listed in the **Table C.1** in the Appendix C.

Significant discrepancy on the displacement proportions exhibited in the negative loading cycles. The flexural portion decreased from about 70% to less than 30% with the increase of ductility and the portion due to panel shear varied between 20%~30%. Larger unaccounted displacement occurred and developed after cycle $DF = 4$ as illustrated by **Figure 5.2.9**, which could be regarded as the sliding displacement but had not been detected by L13.

The discrepancy may be explained by the following reason. Firstly, the rough extents of two ends of the sliding face induced the different potential for each type of displacement to develop under loads in different directions. The left end of the sliding face was flat and just opened at the walls base, whereas the right end of it was protruded above the joint as indicated in **Figure 5.2.7**. The aggregate interlocking, which developed in the compression part of the right end bulge, and the dowel action provided by the vertical bars could arrest the sliding when the wall was loaded in the positive direction. The main portion of the top drift then came from the wall rotation about the centre of compression region. When the wall was loaded in the negative direction, the flat compression sliding face caused the possible sliding displacement to occur in advanced until sufficient dowel action developed. Due to the moment effect, rotation also occurred at the same time, but it was much less compared with that developing under the load in the other direction. High strain developed in the vertical steel bars under sliding because of the kinks in them. Strain hardening developed in the bars so that the wall could reach a higher strength than that developed at the previous ductility level. However, significant strength degradation occurred in the second cycle within a ductility level after $DF = -4$ because the dowel action in the first cycle damaged the concrete around the vertical bars.

The second reason may be that the residual deformation of the steel bars cumulated at the right side of the wall under the negative loading. The sliding displacement developed only after the yield of the compressive steel bars, which occurred firstly at a negative loading at $DF = -4$. This deformation could not be recovered under the reversed loading but increased with the next negative loading.

5.2.6 Moment-curvature relationship

This discrepancy of the component portions in the top drift between two loading directions can also be noted in the variation of the bottom curvature with the moment as illustrated in **Figure 5.2.10**. The positive curvature was much larger than the negative one

at the same ductility, the second cycle of positive curvature was closer than that of first cycle, and due to the sliding influence the second cycle negative curvature was much less than that of the first cycle.

Figure 5.2.11 depicts the wall curvature distribution along the wall height. Only the first cycle at certain displacement ductility level was shown there. It can be observed that the curvature was highly concentrated at the bottom region. The curvature of the region higher than 450 mm from the wall base did not change obviously after $DF = 2$, and the positive and negative curvatures at the same ductility level were close unlike the high discrepancy observed in the bottom of wall.

Figure 5.2.9 Relative portions of the top drift of Specimen S-F1

Figure 5.2.10 Bottom moment-curvature relationship of Specimen S-F1

Figure 5.2.11 Wall curvature distribution of Specimen S-F1

5.2.7 Flange bar strains

Left wall flange

Left flange contained bar grids #A and #B, lying at the left and right side of the flange respectively. Eight mild steel bars (R bar) with a 10 mm diameter were arranged in a vertical direction. This flange was compressed under the negative loading and was tensioned under the positive loading. The details for reinforcement and strain gauge distribution of Specimen S-F1 can be found in **Figure 4.7.1**.

Figure 5.2.12 presents the strain status of #A1 bar, located at the north corner of the left flange. Strain gauges #13, #35, and #53 were distributed along the bar at a level 50 mm,

725 mm and 1235 mm above the base. The bar reached compressive yield at the 8th cycle ($DF = -3$) and at the 15th cycle ($DF = -10$), its bottom strain was abruptly inversed to much higher than its yield strain. Significant tensile strain developed at three height levels of the bar. It was interesting to see that the bottom tensile strain was less than the tensile strain which occurred at 725 mm above the base, where the bar reached tensile yield at $DF = -2$. The reasons might be that the position of the bottom strain gauge was lower than the horizontal crack of the left flange. Due to the initial significant compressive strain, the Bauschinger effect might also have reduced the bottom tensile strain of the bar. Its bottom tensile strain decreased after the 8th cycle, indicating that bond slip occurred in the joint between the wall and the base beam.

Figures 5.2.13 to 5.2.17 present the strain variations versus loading cycles of the other bars within the left flange, including bars #A3, #A7, #B2, #B6 and #B8. Similar behaviour as bar #A1 was observed. Note that most of the bars yielding under compression after $DF = -3$, which had allowed the sliding to develop as mentioned in the previous section. The flange was subject to eccentrically tensile as indicated by the fact that the bars of the north side (#A1 and #B2) developed a larger tensile strain than that of the bars of the south side (#A7 and #B8).

As shown in **Figure 5.2.18**, the stirrups in the bottom of the flange almost yielded at $DF = -8$ when a vertical split crack was observed under high compression.

Right wall flange

The right flange contained reinforcing bar grids #C and #D with bar grid #C laid to the extreme right the wall. This right flange was under tension in the negative cycles and under compression in the positive cycles.

Figures 5.2.19 and 5.2.20 illustrate the strain status of bars #C1 and #D6. The bottoms of these bars experienced high tensile strain at $DF = 2$, and after this ductility level, bars #C1 and #D6 reached the yield strain while significant tensile strains developed at their upper parts. Unlike the bars of the other flange, no compressive yield strain was reached.

5.2.8 Vertical web bar strains

All the web bars, including vertical and horizontal ones, were high yield strength bars (T bars) in Specimen S-F1. They formed two bar grids #S and #N.

Figures 5.2.21 to 5.2.27 depict the strain status of the vertical web bars, including #SV1, #NV4, #SV5, #SV9 and #NV12, distributed from the left to the right of the web as shown in **Figure 4.7.1**. It can be noted that after $DF = \pm 3$, most of the bars reached or surpassed tension yield strain at the bottom under the loading from both directions, except for #NV12 located at 255 mm to the right side of web, where compressive strain developed under the positive load until $DF = 6$.

Under the positive loading, significant tensile strain developed at the medium level (725 mm above the base) of all the bars except for bar #NV12. Bar #NV4 yielded at the middle level as its strain was larger than that of bar #SV1. All of bars, except for #SV1, also develop large tensile strain at this level when the loading reversed, and the yield strain was reached by bars #NV12 and #SV9 there. The upper part of #NV12 was also tensioned close to the yield strain at the high ductility levels as indicated by **Figure 5.2.26**.

It is interesting to see that the highest vertical tensile strain at the middle level of the wall occurred at #NV4 (**Figure 5.2.22**) under the negative loading and occurred at #SV9 (**Figure 5.2.25**) under the positive loading after $DF = 2$, although they were not side bars (#SV1 and #NV12) of the web, unlike predicting by the plane section hypothesis. This may be due to diagonal cracks passing near them as illustrated by **Figure 5.2.29**.

5.2.9 Horizontal web bar Strains

Figures 5.2.27 and 5.2.28 depict the typical strain statues of the horizontal bars. #SH1 and #SH9 bars lay in the positions at 140 mm and 1220 mm above the base respectively. It can be seen that definite tensile strains developed on them after $DF = 2$ when the diagonal crack opened. However, the strain generally was less than the half of the yield strain and it did not increase obviously with the increase of the ductility level because insignificant deformation developed along the diagonal cracks. Some gauges located at the position where the crack passed might reach high tensile strain as illustrated in **Figure 5.2.29**.

Figure 5.2.12 Strain histories in #A1 vertical flange bar

Figure 5.2.13 Strain histories in #A3 vertical flange bar

Figure 5.2.14 Strain histories in #A7 vertical flange bar

Figure 5.2.15 Strain histories in #B2 vertical flange bar

Figure 5.2.16 Strain histories in #B6 vertical flange bar

Figure 5.2.17 Strain histories in #B8 vertical flange bar

Figure 5.2.18 Strain histories in stirrups of left flange

Figure 5.2.19 Strain histories in #C1 vertical flange bar

Figure 5.2.20 Strain histories in #D6 vertical flange bar

Figure 5.2.21 Strain histories in #SV1 vertical web bar

Figure 5.2.22 Strain histories in #NV4 vertical web bar

Figure 5.2.23 Strain histories in #SV5 vertical web bar

Figure 5.2.24 Strain histories in #NV8 vertical bar

Figure 5.2.25 Strain histories in #SV9 vertical web bar

Figure 5.2.26 Strain histories in #NV12 vertical web bar

Figure 5.2.27 Strain histories in #SH1 horizontal web bar

Figure 5.2.28 Strain histories in #SH9 horizontal web bar

Figure 5.2.29 Gauge positions and crack pattern

5.3 Test Results of Specimen S-F2

Specimen S-F2 was the wall with $600 \times 600 \text{ mm}$ irregularly distributed openings as presented in **Figure 4.2.2**. The middle column zone of this specimen was 400 mm wide and the width of the boundary column zone was 200 mm .

5.3.1 Loading history

Figure 5.3.1 depicts the loading history applied to Specimen S-F2. Because the strut-and-tie model, which was a lower bound method, generally under-estimated the ideal strength of the specimen, the yield displacement of the specimen could not be determined. Therefore the yield displacement $\Delta_y = 3.2 \text{ mm}$ of Specimen S-F1 was also applied in its displacement-controlled loading cycles. In the following section, it also demonstrated that the ideal strengths $P_i = -214.2 \text{ kN}$ and $P_i = 220 \text{ kN}$ calculated according to the strut-and-tie models presented in **Figure 4.4.1** and using the measured strength of the steel bars,

Figure 5.3.1 Loading history of Specimen S-F2

could only predicted about 55% of the measured maximum strengths in the test, which were -395.3 kN and 381.2 kN . That means refined strut-and-tie models were required to analyze this wall.

5.3.2 Description of observed behaviour

Loading stage	Description
$0.25P_i \sim 0.5P_i$	No obvious cracks were observed in the web, and only one flexural crack was found in the bottom of each flange at cycle of $0.5P_i$.
$0.75P_i$	One horizontal crack was observed at the upper part of the bottom column in negative loading cycle. When positive loading was applied, horizontal flexural cracks appeared not only in the bottom of the left flange but also in the middle column next to the middle opening. In the middle column zone, some diagonal cracks were observed at the corners of openings. Occurring at the sides of the bottom and middle openings, two horizontal cracks developed along the edges of the lower nodal zone, and went across the whole transverse section of the middle column zone.
$DF = \pm 1$	Due to the relative lower ideal strength was applied, many cracks opened at this ductility. Under the action of the negative loading, two new flexural cracks appeared in the bottom column crossing its whole section as depicted in Figure 5.3.2 (a) . Horizontal flexural cracks also developed at the side of the bottom opening and extended into the bottom panel zone of the web. One of the horizontal cracks, crack ① in Figure 5.3.2(a) , almost extended into the whole section of the panel. Many inclined cracks appeared at the lower corners of the upper opening and extend to the upper corner of the middle opening. A long diagonal crack formed in the upper panel zone. Under the action of the positive loading, more horizontal flexural cracks formed in the left flanges and inclined into the bottom and top panels. A long diagonal crack (crack ②) formed in the bottom panel as shown in the figure. Diagonal cracks near the right lower corner of the middle opening extended and connected with the cracks opened under the negative loading.
$DF = \pm 2$	Horizontal flexural cracks appeared at the middle and the upper regions of the left column zone. The cracks inclined into the near panels as diagonal cracks or extended to the left side of the middle opening. Only one flexural horizontal crack was observed at the right lower corner of the upper opening.

The main diagonal cracks of the upper and middle panel zones, cracks ③, ④ and ⑤ as illustrated in **Figure 5.3.2 (b)**, formed under the action of the lateral loading at this ductility. A long diagonal crack developed in the bottom panel under the negative loading. Horizontal crack ① at the bottom panel extended to the bottom of the right flange, which allowed a sliding face to develop.

$DF = \pm 3$ Large inelastic deformation developed at this ductility. As presented in **Figure 5.3.2 (b)**, more diagonal cracks appeared in the lower side of the upper or middle opening and diagonally inclined into the panels. Many short cracks concentrated in the two middle nodal zones. A crack in the bottom of the right flange opened to a width of 0.4 mm while the crack ③ in the upper panel developed a width of 0.8 mm . Clear concrete struts observed in the panels indicated the load path for the transfer of the lateral loading.

$DF = \pm 4$ As shown in **Figure 5.3.2 (c)**, there were more cracks running parallel to the main diagonal cracks ③ and ⑤ or cracks ② and ④ under the negative or

$DF = \pm 6$ positive loading respectively. New horizontal cracks appeared in the upper parts of the two flanges, including the cracks at the upper corners of the wall. Diagonal cracks developed under the loadings of different directions and were crossed and intersected. The negative diagonal cracks of bottom panels ended at the horizontal crack ①, indicating the sliding developed along this crack. The maximum strengths of both directions were reached at $DF = \pm 4$. Cracks continued widening and extending. A crack at the bottom of right flange reached 5 mm wide.

$DF = \pm 7$ No more new cracks opened in the web in these cycles. Concrete began to crush and spall in the bottom of both flanges and the right lower corner of the

$DF = \pm 8$ bottom panel. Some flange bars buckled at the bottom. Sliding along the crack ① crushed the concrete along the crack as indicated in **Figure 5.3.2 (d)**. The vertical cracks occurred in the bottoms of the flanges when they were under compression. A horizontal crack at the bottom of the right flange developed a width larger than 10 mm under tension. The negative strength of the wall decreased significantly. At $DF = -8$, the wall failed since the strength degraded to less than the 80% of the maximum recorded strength.

After this ductility level, the load increased by two ductility levels until the failure mechanism of wall was clearly shown.

$DF = \pm 10$ As presented in **Figure 5.3.2 (d)**, the concrete in the bottom of the flanges and in the right lower corner of the bottom panel severely crushed and spalled. Most of the steel bars in the flanges buckled and some of them fractured. The diagonal struts in the bottom panel were cut by the sliding face. Though the lateral movement of the wall along the sliding face could be obviously seen, no abrupt strength degradation was observed. **Figure 5.3.2** shows the crushing of the concrete in the bottom of the flanges, **Figure 5.3.3** depicts the crushing of the concrete along the sliding face, and **Figure 5.3.4** displays the crack pattern and damage to Specimen S-F2.

5.3.3 Crack pattern and failure mechanism

Figures 5.3.2 and **5.3.5** present the crack patterns obtained from the test. A good correlation between the crack patterns and the strut-and-tie models shown in **Figure 4.4.1** can be observed. Diagonal cracks in all panel zones indicated that lateral loadings were transferred through these zones by the diagonal struts. Horizontal cracks in the boundary and middle column zones, as well as the vertical cracks in the beam zones, hinted that the tensile forces occurred in these zones.

Though a sliding failure mechanism was clearly demonstrated by the crushing of the concrete along the sliding face, some differences were observed at the failure stage between the behaviour of the two loading directions. Sliding occurred more severely in the positive direction than in the negative direction because the diagonal struts of the bottom panel were cut by the sliding face. It seems that the crushing of the bottom concrete in the bottom of the left flange triggered significant strength degradation to occur under the negative loading. **Figure 5.3.6** displays the failure mode of the wall under the lateral loading in each direction.

(a) At $DF = \pm 1$ ($\delta = \pm 0.125\%$)

(b) At $DF = \pm 3$ ($\delta = \pm 0.375\%$)

Figure 5.3.2 Crack development of Specimen S-F2 (Cont. next page)

(c) At $DF = \pm 6$ ($\delta = \pm 0.75\%$)

(d) At $DF = \pm 14$ ($\delta = \pm 1.75\%$)

(Cont.) Figure 5.3.2 Crack development of Specimen S-F2

Figure 5.3.3 Concrete crushing
in the flange

Figure 5.3.4 Concrete crushing
along sliding face

Figure 5.3.5 Crack pattern and damaged regions of Specimen S-F2

(a) Failure mode under the negative loading

(b) Failure mode under the positive loading

Figure 5.3.6 Failure mechanisms of Specimen S-F2

5.3.4 Lateral load –top displacement relationship

Figure 5.3.7 presents the load-displacement response of Specimen S-F2. Compared with the response of the solid wall Specimen S-F1, the wall with irregular openings achieved similar maximum strength and ductility. The wall failed at the displacement ductility level of $DF = \pm 8$ (drift ratio $\delta = 1\%$) when the maximum strength measured during the cycles were lower than the 80% of the maximum strength obtained in the previous cycles. Large pinching was observed in Specimen S-F2. Though the openings were irregularly distributed, the difference of the maximum lateral loadings of both directions was less than 5%. Before $DF = -8$, the peak strength of the second cycle was higher than 90% of the peak strength of the first cycle. At $DF = -8$ (drift ratio $\delta = 1\%$), concrete crushing at the bottom of the right flange and buckling of the bar triggered a significant drop in strength, and sliding displacement developed obviously after this ductility level. The difference between the two peak strengths of the two cycles increased to about 15%. Though the strength dropped to the failure level, it did not degrade rapidly. According to the peak load and displacement at $DF = \pm 1$, the stiffness of Specimen S-F2 at this loading stage was determined as $K_1 = 59.6 \text{ kN} / \text{mm}$.

Figure 5.3.7 Lateral load-top displacement response of Specimen S-F2

5.3.5 Components of top drift

The contributions of the flexure deflection, the shear deformation and the sliding displacement to the top drift are presented in the **Figure 5.3.8**. The flexural deformation was calculated according to the measurements of LVDTs #L4 & #L7, #L3 & #L6 and #L2 & #L5 as shown in **Figure 4.7.8**. Shear deformation was determined according to the readings of LVDTs #L17 & #L18, and because the sliding face appeared at a level of about 300 mm above the base, the sliding displacement was not successfully measured by the LVDTs assigned to do so. However, the sliding displacement was measured into the LVDTs #17 & #18 distributed along the diagonal directions of the web as a part of the panel shear deformation. Due to this reason, the contribution of shear to the top drift was overestimated. As indicated in **Figure 5.3.9**, the sliding shear displacement Δ_s was averaged into the panel shear at height h_1 and got a panel shear angle γ . The top displacement caused by shear was then estimated by $\gamma(h_1 + h_2)$. As a result, the displacement was amplified by Δ_a . In this case it was about 10% of the sliding displacement.

As illustrated by **Figure 5.3.8**, the portions of the top drift caused by shear and flexure were different under the negative and positive loadings. Under the positive loading, the shear portion was slightly larger than the flexure one and it increased with the ductility, whereas under the negative loading, the flexural part varied from less than the shear part to higher than it as the ductility increased. Before $DF = 8$, the sum of the shear deformation, flexural deformation and sliding displacement was close to the top drift except for that of $DF = 0.75$. After $DF = 8$, the unaccounted parts increased in both directions.

5.3.6 Moment-curvature relationship

Figure 5.3.10 depicts the bottom moment-curvature relationship. It can be noted that the curvature of the negative loading was larger than that of the positive loading at the same ductility level. The distribution of reinforcement which benefited the resistance of the positive flexure might have caused this discrepancy. The concrete crushed at the bottom of the left flange at ductility $DF = -6$ caused the negative moment to significantly drop. Without an increase of the curvature, an abrupt drop of moment from $DF = 7$ to $DF = 8$ indicated that sliding displacement developed obviously at $DF = 8$. A kink of curve at

$DF = 1$ was caused by the fact that a larger displacement than $\Delta_y = 3.2\text{ mm}$ was applied at the positive $0.75P_i$ cycle during the experiment. For the same reason, the strains presented in the following parts were larger at $0.75P_i$ than those of $DF = 1$.

Figures 5.3.11 presents the curvature distribution along the wall height. Similar to the distribution of Specimen S-F1, the curvature concentrated at the bottom of the wall.

Figure 5.3.8 Relative portions of the top drift of Specimen S-F2

Figure 5.3.9 Sliding shear and average panel shear deformation

Figure 5.3.10 Bottom moment-curvature relationship of Specimen S-F2

Figure 5.3.11 Wall curvature distribution of Specimen S-F2

5.3.7 Flange bar strains

The details of the strain gauge distribution of Specimen S-F2 are presented in **Figure 4.7.2**.

Left wall flange

At the left flange of Specimen S-F1, the reinforcing bar grids were numbered as #A and #B with Grid #A being in the left side of the wall. This wall flange was first under compression and then under tension. Three strain gauges were distributed at a vertical bar of grid #A at levels of 50 mm, 1150 mm and 2000 mm above the wall base respectively. The middle and upper strain gauges corresponded to the levels of the centre of the middle and upper panels. In bar grid #B, the middle and upper gauges were located at levels 725 mm and 1575 mm respectively, which corresponded to the middle of the nodal zones.

Figures 5.3.12 to 5.3.14 depict the strain histories versus loading cycles of the vertical bars #A3, #A7 and #B6 respectively. It can be seen that these bars reach tensile yield after cycle 8 ($DF = 3$) at the bottom of the flange. Significant tensile strain also developed in the other two positions after this cycle. The upper two strain gauges data of bars #A7 and #B6 even got close to the yield strain of the steel bars under tension. Certain compressive strain developed in the bottom when the flange was under compression. A little larger tensile strain developed in bar #A7 than that of #A3, which indicated that the tensile force was not evenly distributed in the flange.

Like the stirrups in the left flange of Specimen S-F1, significantly tensile strain developed in the bottom stirrups when the flange was under compression.

Right wall flange

Reinforcing bar grids #C and #D were located in the flange with Grid #C being located in the right side of the wall. The strain gauge levels in grids #C and #D were the same as those in grids #A and #B respectively. The flange was first under tension followed by compression.

Figures 5.3.15 to 5.3.17 display the strain distributions in the flange bars #C3, #C7 and #D2. It can be noted that the bottom and the middle parts of the bars yielded after cycle 7 ($DF = -3$). The upper parts of bars #C3 and #D2 also developed significant tensile strains. Definite compressive strains were measured by the bottom gauges of the bars, but

the compressive strain turned abruptly into tensile ones before the compressive yield strain could be reached.

According to gauge data in the stirrup at the bottom of the right flange, the stirrup almost yielded when the flange was under compression.

5.3.8 Vertical web bar strains

Column zones

There are three column zones in walls with irregular openings, including two boundary column zones and one middle column zone as illustrated in **Figure 3.5.16**. The two boundary column zones consisted of flanges and two parts of the web outside the openings. Web reinforcing bars formed two bar grids #S and #N. #SV1 bar and #SV17 bar were located at the two boundary column zones, while bars #SV7, #SV9 and #SV11 were located in the middle column zones.

Figures 5.3.18 and **5.3.19** present the strain histories of bars #SV1 and #SV17, located at the two boundary column zones. It can be seen that the bottom part of the #SV1 bar yielded at $DF = 3$. The tensile strain of the middle part of this bar increased with ductility and reached yield at $DF = 6$. Definite tensile strain developed at the upper part of this bar. The tensile strain of bar #SV17 developing under the positive loading was less than that of bar #SV1 developing under the negative loading at the same ductility level. It yielded at $DF = -4$.

Figures 5.3.20 to **5.3.22** display the strain histories of bars #SV7, #SV9 and #SV11, located in the middle column zones. The middle and upper gauges of bar #SV7 and #SV9 were distributed at levels corresponding to the centre of the middle and upper panels respectively, while the upper two gauges of bar #SV9 were located at the levels of the centre of the two nodal zones in the middle column zone. According to the strain histories of these bars, all of the bars yielded at the bottom under the positive or negative loading. Large tensile strains also developed at the middles of these bars under the positive and negative loadings. The upper parts of the bars developed obvious tensile strains only under the negative loading when a tie was located at the middle column zone according to the strut-and-tie model presented in **Figure 4.4.1**.

Figure 5.3.23 demonstrates the strain histories of bars #SV3, #NV6 and #NV16 bars. Bars #SV3 and #NV6 were high yield strength bars, located at the bottom panel. Large

tensile strain developed in them under the positive loading. Bar #NV16 which was a mild steel bar and located at the middle panel developed tensile strain under the negative loading.

5.3.9 Horizontal web bar strains

Lower beam zone

Figure 5.3.24 presents the strain histories of bars #SH5 and #SH7. Strain gauges #67 and #68 were distributed in two sides of bar #SH5, while gauge #69 was located at the middle of the bar #SH7. As predicted by the strut-and-tie model presented in **Figure 4.4.1**, large tensile strength developed in them under the positive loading. They developed strains close to yield at $DF = 3$.

Upper beam zone

Figure 5.3.25 displays the strain histories of bars #SH15 and #SH17. Large tensile strain developed under the negative loading also as the tie behaviour of the upper beam zone predicted by the strut-and-tie model.

Panel zones

Figures 5.3.26 and **5.3.27** depict the strain histories of horizontal bars in the panels. Bars #SH1 and #NH4 were located at the bottom panel, while bars #SH11 and #SH21 were located at the middle and upper panels respectively. Significant tensile strains developed in them after $DF = \pm 1$ no matter which direction the loading was applied in. Bar #SH1 reached tensile yield at $DF = 5$.

Figure 5.3.12 Strain histories in #A3 vertical flange bar

Figure 5.3.13 Strain histories in #A7 vertical flange bar

Figure 5.3.14 Strain histories in #B6 vertical flange bar

Figure 5.3.15 Strain histories in #C3 vertical flange bar

Figure 5.3.16 Strain histories in #C7 vertical flange bar

Figure 5.3.17 Strain histories in #D2 vertical flange bar

Figure 5.3.18 Strain histories in #SV1 vertical web bar

Figure 5.3.19 Strain histories in #SV17 vertical web bar

Figure 5.3.20 Strain histories in #SV7 vertical web bar

Figure 5.3.21 Strain histories in #SV9 vertical web bar

Figure 5.3.22 Strain histories in #SV11 vertical web bar

Figure 5.3.23 Strain histories in web panel vertical bars

Figure 5.3.24 Strain histories in lower beam zone horizontal bars

Figure 5.3.25 Strain histories in upper beam zone horizontal bars

Figure 5.3.26 Strain histories in lower panel horizontal bars

Figure 5.3.27 Strain histories in middle and upper panel horizontal bars

5.4 Test Results of Specimen S-F3

Like Specimen S-F2, Specimen S-F3 also has $600 \times 600 \text{ mm}$ irregularly distributed openings but the horizontal stagger between the two openings is 200 mm , while the openings were 300 mm away from the boundaries. The details for Specimen S-F3 are presented in **Figure 4.2.3**.

5.4.1 Loading history

Figure 5.4.1 depicts the loading history applied to the wall specimen in the experiment. It is the same as the loading history applied to Specimen S-F2, except that the ideal lateral loadings $P_i = -214.2 \text{ kN}$ and $P_i = 203.3 \text{ kN}$ were applied, which were estimated by the strut-and-tie models illustrated in **Figure 4.4.2**. When the displacement ductility factor became greater than $DF = \pm 1$, two loading cycles were applied. Before $DF = \pm 8$ the ductility factor increased by 1, while after that the ductility factor increased by 2 until the structure was loaded to failure.

Figure 5.4.1 Loading history of Specimen S-F3

5.4.2 Description of observed behaviour

Loading stage	Description
$0.25P_i$ $\sim 0.5P_i$	No obvious cracks were observed in the web except a flexural crack was found in the bottom of the right flange.
$0.75P_i$	Flexural cracks formed in the two flanges under the action of reversed cyclic loadings. Most of them were concentrated in the lower half parts of the flanges. All cracks in the middle parts of the right flange extended to the upper side of the bottom opening. Three cracks in the lower part of the left flange extended into the bottom panel zone as inclined cracks. One of them, located at about 400 mm above the base, sloped to the base of the wall. A horizontal crack appeared at the top of the middle column and extended to the upper left corner of the middle opening. No cracks were detected in the middle and upper panels.
$DF = \pm 1$	Under the negative loading, only one horizontal crack was added to the upper part of the left flange at the upper beam zone level. When the positive loading was applied, vertical cracks opened at the centre of the sides of the middle and bottom openings and extended into the lower beam zone, as illustrated in Figure 5.4.2 (a) . This indicated that the lower beam zone was under tension when the positive loading was applied.
$DF = \pm 2$	As displayed in Figure 5.4.2 (a) , many flexural horizontal cracks formed in the middle parts of the flanges. The new cracks developing in the left flange extended into the boundary of the openings. The cracks in the right flange extended into the middle panel zone and formed the diagonal cracks, including the main diagonal crack ③ in this panel. Vertical cracks appearing in the middle of the upper beam zones indicated that the tensile force was transferred through this zone under the negative loading. Inclined cracks ① and ② formed in the bottom panel under the action of the negative and positive loadings respectively. Two horizontal flexural cracks developed in the lower part of the middle column zone and more flexural cracks appeared at the lower part of the right column zone under the negative loading.

- $DF = \pm 3$ Under the negative loading, flexural cracks formed in the lower part of the middle column zone with two of them inclining into the bottom panel and extending to the base of the wall. A diagonal crack developed parallel to crack ①. More inclined cracks appearing in the upper nodal zone, connected the two corners of the openings. Cracks developed at the previous ductility levels kept on extending. Under the action of the positive loading, the main diagonal crack ④, shown in **Figure 5.4.2 (b)**, opened in the middle panel. Horizontal cracks developing at the middle part of the middle column zone indicated that a vertical tensile force existed at this zone. A crack in the bottom of the left flange reached 2 mm.
- $DF = \pm 4$ As demonstrated in **Figure 5.4.2 (b)**, more diagonal cracks formed at the bottom and middle panel zones under the negative loading. Diagonal crack ⑤ formed in the upper panel zone when the positive loading was applied to the wall, and more diagonal cracks opened in directions parallel to the main cracks in the middle and bottom panels. Cracks developing in the positive and negative loadings crossed one another in the panels. The cracks in the bottom panel which developed under the action of the positive loading formed a clear fan-shaped crack pattern. More horizontal cracks appeared in the middle column zone. No other flexural cracks opened in the flanges.
- $DF = \pm 5$ As displayed in **Figure 5.4.2 (c)**, some new diagonal cracks, including crack ⑥, formed in the upper panel zone under the negative loading at these
- $DF = \pm 7$ ductility levels. In the other panel zones, new short diagonal cracks opened within the struts formed in the previous cycles. Existing diagonal cracks continued to widen and extend. The crack at the bottom of the left flange reached 10 mm at $DF = -7$. The maximum negative and positive loadings were reached at $DF = \pm 6$. The concrete at the left bottom corner of the wall began crushing at $DF = \pm 7$.
- $DF = \pm 8$ As shown in **Figure 5.4.2 (d)**, no new cracks opened during these cycles. The
- $DF = \pm 10$ concretes in the bottoms of the flanges and at the two lower corners of the web continued to crush and spall, while the steel bars in the flanges buckled, and pulled out under the action of the cyclic loading. **Figure 5.4.3** presents

the buckled bars in the right flange.

$DF = \pm 12$ The concrete severely crushed and spalled at the bottom. Most of the steel bars in the flanges buckled and some of them fractured. The structural failure occurred at $DF = \pm 14$ ($\delta = 1.75\%$) when the peak negative loading was less than 80% of the maximum negative loading. **Figure 5.4.4** displays the concrete crushing status in the bottom panel, while **Figure 5.4.5** depicts the crack pattern and failure zones.

5.4.3 Crack pattern and failure mechanism

The crack pattern of Specimen S-F2 presented in **Figure 5.4.5** exhibits a good correlation with the strut-and-tie models displayed in **Figure 4.4.2**. The diagonal cracks in the panel zones indicated the diagonal struts existing in these zones, while the horizontal or vertical cracks formed in the column or beam zones indicated the ties developing in these zones.

As depicted in **Figure 5.4.6**, a typical flexural failure occurred in this specimen. The failure was triggered by the crushing of the concrete in the two lower corners of the walls under the action of the reversed cyclical loading. Buckling and fracturing of the steel bars were observed after $DF = \pm 8$ ($\delta = 1\%$). Compared with the failure mechanism under the positive loading, the failure mode under the negative loading was closer to the flexural failure of a solid wall.

5.4.4 Lateral load-top displacement relationship

Figure 5.4.7 demonstrates the load-displacement response of Specimen S-F3. It can be noted that the wall could achieve satisfactory strength and even reach a higher drift ratio compared with Specimen S-F2. The strut-and-tie models presented in **Figure 4.4.2** also failed to estimate the maximum strengths of this wall as that for Specimen S-F2. No abrupt drop of strength was observed during the loading cycle before failure. The peak loading of the second cycle was very close to that of the first cycle at the same displacement ductility level before $DF = \pm 10$. The difference was only about 5%. The pinching was close to that of a solid wall and was better than that of Specimen S-F2. At $DF = \pm 1$, its stiffness is $K_1 = 52.60 \text{ kN/mm}$. The maximum ductility level of this wall ($DF = \pm 14$) was also determined to be higher than its actual level because the low yield displacement $\Delta_y = 3.2 \text{ mm}$ was applied.

(a) At $DF = \pm 2$ ($\delta = \pm 0.25\%$)

(b) At $DF = \pm 4$ ($\delta = \pm 0.5\%$)

Figure 5.4.2 Crack development of Specimen S-F3 (Cont. next page)

(c) At $DF = \pm 6$ ($\delta = \pm 0.75\%$)

(d) At $DF = \pm 10$ ($\delta = \pm 1.25\%$)

(Cont.) Figure 5.4.2 Crack development of Specimen S-F3

Figure 5.4.3 Buckling of the bars

Figure 5.4.4 Concrete crushing in the bottom panel

Figure 5.4.5 Crack pattern and failure zones of Specimen S-F3

(a) Failure mode under the negative loading

(b) Failure mode under the positive loading

Figure 5.4.6 Failure mechanisms of Specimen S-F3

Figure 5.4.7 Lateral load-top displacement response of Specimen S-F3

5.4.5 Components of top drift

As illustrated in **Figure 4.7.9**, three groups of LVDTs, including #L4 & #L7, #L3 & #L6, and #L2 & #L5, were used to measure the flexural deformation along the vertical direction of the wall. LVDTs #L17 and #L18 were applied to measure the global shear deformation of the whole wall panel.

Figure 5.4.8 shows the relative portions of the flexure and shear deformations in the top drift. It indicates that the flexure was dominating in the negative top drift. It was larger than 60% and increased with the displacement ductility level. In the positive top drift, the shear deformation contributed at least 40% and reached maximum at $DF = 6$.

5.4.6 Moment-curvature relationship

Figure 5.4.9 displays the bottom moment-curvature relationship. The negative curvature was slightly higher than the positive one at the same ductility level. The negative curvature of the second cycle was close to that of the first cycle, and the difference of

curvatures of the two cycles at the same ductility level was higher under the positive loading. According to the curve, the wall entered a flexural inelastic range after $DF = \pm 3$.

Figure 5.4.10 presents the curvature distribution of the wall along the wall height. Similar with other specimens, the flexural deformation concentrated in the bottom of the wall.

Figure 5.4.8 Relative portions of the top drift of Specimen S-F3

Figure 5.4.9 Bottom moment-curvature relationship of Specimen S-F3

Figure 5.4.10 Wall curvature distribution of Specimen S-F3

5.4.7 Flange bar strains

Left wall flange

As illustrated in **Figure 4.7.3**, the left wall flange contained reinforcing bar grids #A and #B with the bar grid #A being located at the left side of the wall. The middle and upper strain gauges at grid #A were distributed at the levels according to the centre of the middle and upper panels respectively, while the two strain gauges in grid #B were located at the levels of the centre of the two nodal zones respectively. The flange was first compressed and then was tensioned.

Figures 5.4.11 to 5.4.14 depict the strain histories of vertical bars #A1, #A3, #A7 and #B6. Observe that all bars except for bar #A1 reached tensile yield strain at the bottom after cycle 5 ($DF = 2$) under the action of the positive loading. The middle parts of the bars could develop tensile strains close to yield at cycle 9 ($DF = 3$). Definite tensile strain developed in the upper parts of the bars and significant compressive strains were detected in the bottom of all bars under the negative loading. The bottom part of bar #A1 even yielded under compression at cycle 5 ($DF = -2$).

Like the stirrup strains in the previous specimens, the stirrups in the bottom of the two flanges could develop certain tensile strains when the flanges were under compression according to the strain gauge measurements.

Right wall flange

The right wall flange contained bar grids #C and #D. Grid #C was located in the right side of the wall and the strain gauge levels of grids #C and #D were the same as those of grids #A and #B respectively.

Figures 5.4.15 and 5.4.16 present the strain histories of bars #C3 and #D2 respectively. Bar #D2 was located at the north side of flange. As illustrated in the **Figure 5.4.16**, the bar #D2 developed a tensile strain much higher than the tensile yield strain at cycle 5 ($DF = 2$). Its middle and upper parts also developed strains close to the yield strain. Significant compressive strain was observed in its bottom part when flange was under compression. Similar behaviour could be observed in bar #C3 and in the other vertical bars of the flanges.

5.4.8 Vertical web bar strains

Column zones

Bars #SV1 and #SV17 were located at the west and east sides of the web respectively and they were belonged to the two boundary column zones.

Figure 5.4.17 depicts the strain variations of the deformed bar #SV1 with loading cycles. The figure shows that the bottom of the bar yielded under tension at cycle 7 ($DF = 3$). After cycle 11 ($DF = 5$), this part of the bar entered a strain hardened state according to the gauge reading. Significant tensile strength developed in the middle part of the bar whereas no obvious strain was detected in the upper part of the bar.

Figure 5.4.18 presents the strain histories of the deformed bar #SV17 on the other side of the web. It can be noted that it yielded under tension at cycle -7 ($DF = -3$) when the negative loading was applied. Large tensile strain was also observed at the middle part of the bar. After cycle 9 ($DF = 4$), the bottom part of the bar also developed large tensile strain under the positive loading. No compressive strain developed in this bar.

Middle column zone

Deformed bars #SV7, #SV9 and #SV11 were located at the middle column zone. The strain histories of these bars were plotted in the **Figures 5.4.19 to 5.4.21** respectively. Similar strain distributions were observed among them. Under the action of the negative loading, large tensile strains developed at all heights of the bars, while under the action of the positive loading, tensile strains only developed at the bottom and middle parts of the bars.

Panel zones

Figure 5.4.22 presents the strain histories of bars #SV3, #NV6 and #SV13. Bars #SV3 and #NV6 located in the bottom panels, were high yield strength deformed bars. Bar #SV13 was a plain mild steel bar, located at the middle panel zone. It indicates that the two deformed bars in the bottom panel yielded in tension under the positive loading after cycle 5 ($DF = 2$) and under the negative loading after cycle 9 ($DF = -4$). Plain bar #SV13 developed definite tensile strain under the action of the positive loading.

The strain histories of other vertical bars in the middle and upper panel zones illustrated that these bars only developed small tensile strains, which were less than 0.0012 like that of bar #SV13.

5.4.9 Horizontal web bar strains

Lower beam zone

Figure 5.4.23 plots the strain histories of bars #SH5 and #NH8, located at the lower beam zone. As predicted by the strut-and-tie model presented in **Figure 4.4.2**, the bars developed significant tensile strain under the positive loading. The right part of the bar #SH5 reached tensile yield strain after cycle 15 ($DF = 7$).

Upper beam zone

Deformed bars #SH15 and #SH17 were located at the upper beam zone. According to the **Figure 5.4.24**, these bars developed a tensile strain when the wall was subjected to the negative loading as predicted by the strut-and-tie model, but the tensile strain was less than the yield strain.

Panel zones

Figure 5.4.25 presents the strain histories of plain bars #SH1 and #NH4, located in the bottom panel. It can be seen that these bars yielded under tension after $DF = -6$ when the wall was subjected to negative loading. Bar #NH4 also developed a tensile strain up to yield under the action of positive loading after cycle 9 ($DF = 4$).

The strains histories of plain bars #SH11 and #SH21, located in the middle and upper panel zones respectively, are displayed in **Figure 5.4.26**. Definite tensile strain developed in bar #SH11 under the positive loading, while bar #SH21 developed a definite tensile strain under the negative loading.

Figure 5.4.11 Strain histories in #A1 vertical flange bar

Figure 5.4.12 Strain histories in #A3 vertical flange bar

Figure 5.4.13 Strain histories in #A7 vertical flange bar

Figure 5.4.14 Strain histories in #B6 vertical flange bar