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PRECAST SHEAR WALL WITH HORIZONTAL CONNECTION

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School of Civil and Environmental Engineering

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Han Hongsheng

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SUMMARY

Shear walls are widely used in concrete structures to resist lateral force originated from wind and earthquake. Currently, shear walls using steel sections as flexural reinforcement at two ends are successfully used in practice, and this kind structure also showed good performance in past severe earthquake. Some provisions are available in the NEHRP (1997) and design recommendations also have been implemented in AIJ (1985).

In current construction practice of this kind of shear wall, the steel frame must be erected first, and then concrete is placed. This sequence may cause stability problem of the steel frame before concrete is cast and vertical formworks are also needed. In order to make a breakthrough in this field and accelerate the construction speed on site without influencing the performance of the wall, the concept of "precast shear wall" with effective connection is introduced into this kind of wall in present research.

In order to investigate performance of precast walls with horizontal connection, six shear walls, two monolithic walls acting as prototypes and four precast shear walls serving as counterparts, have been tested. Two kinds of steel sections, channels and I-Beams have been used in these walls serving as flexural reinforcement. The influence of number of shear connectors in the horizontal connection on the behavior has also been studied.

All specimens have been tested under reversed cyclic load. Similar ultimate strength, drift ratio were observed between these monolithic walls and their precast counterparts in the experiment. However, meantime monolithic wall exhibited different features from those of precast ones during loading course, including

cracking patterns, failure modes and failure locations, internal force paths, stiffness and capacity of energy dissipation.

Theoretical model based on macro model was developed. The precast walls are divided into macro model for walls and for connection with different material models. The suggested macro model produced reasonable good prediction for the precast walls and a parametric study based on the macro model has been done

Based on cracking patterns and records of strain gauges placed horizontal and vertical reinforcement, strut-and-tie models of each specimen during different loading stages have been developed.

A nonlinear finite element program, DIANA 8.0, has been used to study the behavior of specimens under cyclic loads. Two dimensional and three dimensional models have been built to simulate behavior of these specimens. In two dimensional models, steel sections, channels and I-Beams, were modeled by truss element model. It showed that behavior of specimens with embedded channels could be captured by both two and three dimensional models. However, the ultimate strength of wall with I-beams is underestimated 15% by two-dimensional model. In three dimensional models, the steel sections were simulated by shell elements and the interface action between concrete and steel sections was simulated by interface elements with frictional material model. The yield strength, ultimate and ductility were predicted with a high agreement by three dimensional models for all walls. Linear and nonlinear frictional material models were also investigated.

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LIST OF SYMBOLS

A	total area of the element
A_{cs}	effective cross sectional area of strut
A_e	effective area of steel section
A_s	sectional area
A_{sd}	area of the diagonal reinforcement in diagonal direction
A_{sfl}	area of edge column longitudinal reinforcement
A_{sp}	cross sectional area of outer spring
A_{str}	effective area of diagonal strut
A_w	net area of the concrete section bounded by the web thickness t_w and the length of the section in the direction of the shear force l_w
A_v	area of the shear reinforcement within distance s
a_s	depth of the diagonal strut
b	thickness of wall
b_h	the width of the hoops
b_s	width of the diagonal strut that can be taken as the width of the wall web
b_v	effective web width
b_w	thickness of web
C	cohesion
C_c	compression force taken by concrete
C_s	compression force taken by steel
c	depth of neutral axis, cohesion
c_1, c_2	parameters in definition of tensile behavior of concrete
D	matrix of stiffness
d	effective compression block
d_1, d_2	distance from centroid of steel section and that of bars in tension zone to the extreme compression fiber
d_{nm}	stiffness in normal direction
d_{ss}	stiffness in tangent direction
d_v	effective shear depth
d	length of lever arm
E	modulus of elasticity
E_c	concrete modulus of elasticity
E_s	steel modulus of elasticity
F	horizontal force acting on top beam
F_h	sum of forces in horizontal reinforcement

F_v	sum of forces in vertical reinforcement in central 0.8 width of wall
F_i	force in strut or tie i
F_s	force in the bar or group of bars
F_{sp}	force in outer spring,
F_y	tension force on steel sections when the specimens reaches its yield strength
$f(\sigma, \kappa)$	principal stress space
f_b	bond stress
f_{bu}	design ultimate anchorage bond stress
f'_c, f_c	compression strength of concrete cylinder
f_{rd}	the design strength of concrete
f_{cu}	compression strength of concrete cube
f_d	stress in diagonal reinforcement
f_e	elastic strength
f_{sp}	stress in spring
f_t	tensile strength of concrete
f_u	tensile strength of reinforcement
f_y	steel yield strength
f_{yd}	design strength of this reinforcement
G	shear modulus, shear stiffness
G_f^I	tensile energy of concrete element.
G^{cr}	shear stiffness of cracked concrete
g	acceleration due to gravity
$g(\sigma, \kappa)$	plastic potential
H_i	distance from center of interval of wall to the horizontal axis of the top beam
H_w	the height of the wall measured from base line of wall to the horizontal axial of the top beam
h	wall height, crack bandwidth
h_1	distance from load point to center of connection
h_2	distance from load point to top of macro wall model
h_c	distance from central line to horizontal connection, from rotation center to the top of wall.
h_s	mean story height
h, h_w	wall height
h_i	the height of interval covering horizontal connection
I	important factor
j	stress block depth factor
K_1	the axial stiffness

K, K_i	initial stiffness
$K_{sp,e}$	Elastic stiffness of outer spring
k	internal state variable, empirical factor
l_0	the element's length from the point of zero moment, to the point of maximum moment
l	width of wall, anchorage length of reinforcement, action length of spring
l_e	elastic action length of bond
l_p	plastic action length of bond
$l_{d,sp}$	action length of diagonal reinforcement
l_w	width of wall
l_i	distance between the inclined bars' center axes, at the base cross the section
l_p	length of the plastic hinge
M_{sd}	the design moment
m	modular ratio of steel to concrete
N	vertical force acting on the wall
N_{sd}	the design axial force at the potential sliding plane
n	number of bars
P	wall axial load, resistance
P_i	ideal strength
P_y	lateral force acting on top beam when specimens reaches its yielding strength
Q	normal load
S_o	overstrength
S_u	design strength
S_y	yield strength of member
s	spacing of shear reinforcement
s_h	the spacing of the hoops
T	tensile strength of brittle material
T_0	tensile limit, a tensile limit is used when the tensile strength of the material is lower than the strength defined by the failure criterion
T_1	tensile strength of vertical web reinforcement and the dowel action of the horizontal web reinforcement
T_c	compressive force at the edge column, concrete tie in tension without reinforcement
T_{cfr}	aggregate interlock and friction resistance force at the edge column
T_d	dowel action strength of the horizontal web reinforcement,
T_{dg}	tensile strength of bidiagonal reinforcement
T_{fr}	aggregate interlock and friction resistance force, along the crack in the web,
T_h	tensile strength of the horizontal web reinforcement and dowel action of vertical web reinforcement

T_s	tensile force in steel, ties in tension with reinforcement
T_{s-bars}	tensile force by bars
$T_{s-channels}$	tensile force by channel
t	thickness of shell element, traction
t_0	tolerance between bolts and bolt holes
t_n	normal traction
t_t	shear traction, tolerance for bolts
t_w	web thickness
u_b	the bond strength
u_s	horizontal displacement of top beam
V_c, V_n, V_u	shear strength of member by concrete
V_{dd}	dowel action of vertical bars
V_{fd}	the friction resistance
V_{id}	the shear resistance of diagonal bars
V_{cd}	concrete contribution to shear strength against tension failure
V_{Rd3}	shear strength against diagonal tension failure
V_{wd}	web reinforcement contribution to shear strength against tension failure
z	effective internal lever arm
α	interval length of wall, factors to account for reinforcement
	overstrength, angle between diagonal and horizontal
α_s	shear span ratio, depth of equivalent rectangular stress block, empirical shear reductor
β	coefficient dependent on the bar type, shear retention factor, dilation angle, inclined angle of teeth
δ_h	horizontal displacement
δ_{hp}	horizontal shear deformation
δ_u	roof displacement
δ_v	vertical displacement
ε_0	concrete strain at maximum compressive strength
ε_1	principal tensile strain
ε_2	principal compression strain
ε_{50u}	strain at 50% strength on the descending branches of the unconfined concrete
ε_{50h}	strain at 50% strength on the descending branches of the confined concrete
$\varepsilon_{c,max}, \varepsilon_{cu}$	extreme fiber concrete compression strain
ε^e	elastic strain

ε_{mi}	mean strain of a member i
ε_{mc}	maximum compression strain in connection concrete when specimen reaches its ultimate strength
$\varepsilon_{nm,ult}^{cr}$	ultimate crack strain
ε_{nm}^{cr}	crack strain.
$\dot{\varepsilon}^p$	plastic strain rate
$\varepsilon_{sy}, \varepsilon_y$	yielding strain of steel reinforcement in tension
ε_{yr}	tensile strain in connection steel when the wall reach its yield strength
ϕ	diagonal reinforcement inclined angle, curvature, the diameter of the largest steel bar, internal frictional angle
ϕ_o	safety factor between shear strength and flexural strength, initial angle of internal friction
ϕ_{ss}	steady state frictional angle
ϕ_i	curvature at horizontal connection due to tolerance between bolts and bolt holes
ϕ_u	friction angle between the two materials
ϕ_p	post-yield curvature
ϕ_y	yield curvature
γ	factors to account for reinforcement strain hardening, shear strain
γ_{sy}	ratio of yield strength of precast walls to that of their monolithic counterpart
$\gamma_{\Delta y}$	ratio of yield displacement of precast walls to that of their monolithic counterparts
γ_{su}	ratio of ultimate strength of precast walls to that of their monolithic counterparts
$\gamma_{\Delta u}$	ratio of ultimate displacement of precast walls to that of their monolithic counterparts
φ	angle between tensile strength and horizontal line, dilation angle
φ_e	the effective bar size which, for a single bar is equal to the bar size and, for a group of bars in contact is equal to the diameter of a bar of equal total area
μ_p	curvature ductility
μ	ductility factor defined as the ratio of Δ_u/Δ_y , friction coefficient
μ_f	the concrete-to-concrete friction coefficient under cyclic action
ν	nominal compression stress, Poisson's ratio
θ	inclination angle of the crack, rotation of intervals, angle between diagonal strut and tie
ρ	tension steel reinforcing ratio

List of Symbols

ρ'	compression steel reinforcing ratio
ρ''	distributed steel reinforcing ratio
ρ_1	ratio of tension reinforcement
ρ_s	confinement reinforcement ratio
ρ_h, ρ_v	ratios of horizontal and vertical web reinforcement, respectively
ρ_w	shear reinforcement
ξ	the normalized neutral axis depth
$\dot{\lambda}$	plastic multiplier
λ_1	the ratio of actual to specified yield strength
λ_2	the potential increase resulting from strain hardening
σ	normal stress
σ_2	principal compression stress
$\sigma_{d,max}$	maximum compressive stress acting on the nodal zone
$\bar{\sigma}(\kappa)$	uniaxial yield strength of concrete as a function of the internal state variable κ .
τ	peak state of shear stress in friction model
τ_{Rd3}	diagonal tension stress,
τ_{cd}	stress contribution by concrete to τ_{Rd3} ,
τ_{wd}	stress contribution by web reinforcement to τ_{Rd3} ,
ζ	softening coefficient
Δ	deformation
$\Delta v_{i,j}$	vertical deformations
Δ_y	yield displacement
Δ_u	ultimate displacement of specimen at failure
Δu_n	opening component of interface cracks
Δu_t	sliding component of interface cracks
$\sum A_{sj}$	the sum of the areas of the vertical bars in the web and of purposely arranged bars in the boundary elements of the wall
$\sum A_{st}$	the sum of the areas of all diagonal bars in both directions

CHAPTER 1

INTRODUCTION

1.1 Problem Definition

Shear walls which have shown good performance over the past forty years during severe earthquakes are widely used in concrete structures to resist lateral loads originating from earthquakes and wind. Composite shear walls, namely steel reinforced concrete (SRC) shear walls, which incorporate steel sections and concrete, have been successfully used in the First City Tower in Houston recorded by Taranath (1985). Today, the manner in which steel reinforced concrete is used is based on the construction sequence, because the steel frame must be erected before the concrete is placed. This construction manner requires vertical formwork which creates potential problems with instability of the steel frame without the concrete placed. As a consequence construction speed can be reduced. However, construction speed can be increased if the shear wall is cut into several segments in the vertical direction and prefabricated in a plant, and connected in an effective manner on site.

Extensive studies have been made regarding the seismic response of reinforced concrete shear walls. Research on steel frame-reinforced concrete infill systems was conducted by Makino et al. (1980), Makino (1984), and Tong (2001). Yamada (1990) conducted experiments on composite frame systems including composite shear walls. Tupper (1999) studied reinforced concrete walls with steel boundary elements. The bond between the concrete and steel section in the composite structure was previously assumed to be the same as that of steel reinforcement and concrete by some researchers. However, because of the difference of surface conditions between the steel section encased in concrete and the deformed rebar, the

bond slip curve of the deformed reinforcement, which is a critical parameter, may not be directly used in finite element method (FEM) analysis of composite members. Although design recommendations for seismic design of composite systems have been implemented in Japan (AIJ, 1985) and initial design provisions for the composite wall systems have been proposed in NEHRP (1997), this kind of system is not widely used in practice all over the world due to lack of sufficient research. Makino et al. (1980) and Makino (1984) showed that two buildings of this type behaved satisfactorily during the Great Kanto Earthquake of 1923 in Japan, while other buildings were badly damaged.

When the concept of “precast” is introduced to composite walls, that is, when the wall is cut into several vertical segments and connected with horizontal connections between the segments, the force transfer mechanism may change a lot from the original. Therefore, experimental and theoretical research programs are needed to investigate the behavior of precast composite wall system and initiate the development of appropriate design methodology.

The strut-and-tie models first proposed by Ritter (1899) and Morsch (1902) have been widely used as powerful tools in the analysis of reinforced concrete structures. The strut-and-tie model is a unified approach that considers all load effects including moment, axial force, shear and torsion, simultaneously. It has evolved as one of the most useful design methods for shear critical structures and for disturbed regions in concrete structures. The strut-and-tie model provides a rational approach to represent a complex structural member with appropriate simplified truss models. Now as a design method for deep beams and shear walls, the strut-and-tie model is included in AASHTO LRFD Specifications (1994) and the ACI-318 code (2005). Strut-and-tie models are developed for the specimens in this research for different loading stages.

In current seismic design of reinforced concrete structures, performance-based design is mainly used, in which the seismic load is directly related to the performance of the structures. Two performance-based designs, capacity design and displacement-based design, have been proposed and verified as more efficient in the seismic design of reinforced concrete structures than strength-based design method.

1.2 Objective and Scope of Research

The objective of this research was to investigate the seismic performance of composite precast shear walls with horizontal connections in which transfer of forces is simple and the force can be easily quantified, and to develop related design recommendations.

Different from previous research by US PRESSS and Soudki (1994) on precast wall panels with reinforcing bars, the precast walls in this study were walls with embedded steel sections, channels and I-beams used as a solution for reinforcement congestion. To simplify assemble operation on site and make it practicable at a high position of the building, where the quality of welding is difficult to be guaranteed, bolting connection, a kind of the mechanical connection, was applied in this research. This will ease the erection of the wall panels without additional props. Shear connectors were widely studied in vertical connection in US PRESSS. In Soudki (1994) study shear key or friction between panels were used to resist shear force along the horizontal connection. In present research, mechanical shear connectors were used in the horizontal connection and the effect of number of shear connectors on the performance of the whole horizontal connections was also investigated in case that the shear force was too high to cause local failure of wall panel.

This research consisted of three parts. The first part was the design, manufacturing and testing six specimens including two monolithic walls and four precast walls. Characteristics of these walls including strength, stiffness, ductility and energy displacement were studied. To predict general behavior of precast walls with horizontal connections, the author developed a wall model based on macro model for precast walls with different material models for connection zone and wall panel. The second part was the development of strut-and-tie models for the specimens at different loading stages, which provided an explanation for the force transfer and failure modes and failure zones. The third part was the application of that finite element analysis to study the behavior of these specimens. This part also involved parametric study to investigate the effect of some parameters on the behavior of the walls. These parameters included the types of horizontal connection, shape of the steel sections, and behavior of the interface between the steel section and concrete.

1.3 Organization of Thesis

This thesis consists seven chapters. Chapter 1 is an introduction to the background, objective and scope of this research. Chapter 2 provides a literature review of reinforced concrete shear walls and composite shear walls, shear connections, and models of reinforced concrete shear walls including strut-and-tie models. In Chapter 3, the design, manufacturing and testing of six specimens, two monolithic shear walls acting as prototypes and four precast shear walls with horizontal connections acting as counterparts were are described. In Chapter 4, seismic performance of these tested specimens was evaluated through strength, stiffness, ductility and energy dissipation. A theoretical model based on macro model is developed for precast walls and parametric study based on the developed model has been done. In Chapter 5, based on development of cracking patterns in

different loading stages and records of strain located on steel sections and reinforcement, strut-and-tie models were developed. In Chapter 6, the analysis using commercial finite element software DIANA, finite element analyses is described. In Chapter 7, conclusions and design recommendations are given.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Over the last forty years, shear walls have exhibited good performance in severe earthquakes in which modern reinforced concrete buildings have stood the test of violent shaking (Fintel 1991, 1995). With increasing recognition of the benefits of composite structures, composite shear walls using steel sections as longitudinal reinforcement have been widely developed and studied.

In an attempt to break through the construction sequence, and make composite structures more flexible, the concept of “precast” will be introduced into this field. Because shear and bending moment exist simultaneously at any horizontal section of the shear wall, the connections between the two parts must be able to transfer the shear and moment reliably.

In order to rationally analyze the behavior of the reinforced concrete members, strut-and-tie models, using concrete struts to resist compression and reinforcement ties to carry tension, were proposed by Ritter (1899) and Morsch (1902). In this chapter, past research on the behavior of shear walls and composite shear walls are reviewed. Research on shear connections including the effect of the diagonal reinforcement on the shear behavior of shear walls is also reviewed. Models (strut-and-tie and wall member models) used in analyzing behavior of shear walls are also presented. At the end of this chapter the philosophy of seismic design is briefly introduced.

2.2 Reinforced Concrete Shear Walls

The evolution of the modern approach to earthquake engineering of buildings started in the 1950s, at a time of intense construction after the Second World War. Most of the analytical research in the 1950s and 1960s on the response of structures to earthquakes has emphasized the importance of using ductile moment-resistant frames to reduce seismic forces. Presupposing higher seismic forces in more rigid structures, and assuming brittle response of shear walls to in-plane lateral forces, it has been concluded that severe damage could be expected in shear wall buildings. Based on this thinking, shear walls were considered undesirable for earthquake resistance at that time. In the meantime, it was found that it is better and more efficient to incorporate shear walls for structural systems of multistory structures of concrete resisting wind, (Fintel 1991 and 1995). It was found by Paulay (1980, 1986) shear wall could work in a relative easy and stable way to dissipate energy if a few special detail requirements were met.

Fintel (1991, 1995) investigated and reported on the behavior of modern structures in a dozen earthquakes throughout the world since 1963, and was not aware of single concrete building containing shear walls that had collapsed. The conclusion was buildings containing shear walls, even if only conventionally reinforced, withstood severe earthquakes, mostly without damage.

Experiments investigating the behavior of reinforced concrete (RC) walls under earthquake loading have been conducted mainly in the U.S., Japan, New Zealand, and Europe. The pioneers in research on RC walls were Benjamin and Williams (1957), who tested squat walls. Two reports by Regan (1971) to CIRIA included a significant amount of state-of-the-art research on shear in the U.S. and U.K.

Experimental data from Oesterle et al. (1979) demonstrated that increasing the amount of horizontal web reinforcement was not sufficient to avoid the mode of shear failure. This experiment showed that increasing the amount of horizontal web reinforcement by a factor of two had a negligible influence on the hysteretic response of these walls.

Oesterle (1980) found that in ductile structural walls, the shear deformation in the plastic region may be significantly larger than in other, predominantly elastic, regions. Shear resistance of reinforced concrete mainly comes from elastic region of the walls under cyclic loads suggested by Corley (1981), which gives implication of the layout the vertical reinforcement.

The flexural response and effect of reinforcement distribution on behavior of squat walls were studied by Paulay (1975). The mechanism of shear resistance was studied by Barda (1972). It showed that a significant portion of the shear acting on the top of the squat wall was directly transmitted to the bottom by diagonal compression. Failure modes and performance-improving methods including diagonal reinforcement were widely studied by Priestley and Paulay (1982).

In order to investigate the effect of parameters such as the height-to-width ratio, the axial load, the concrete strength, and the amount of web horizontal reinforcement on wall behavior, thirteen large-scale wall models were tested under the combined action of a constant axial load and a horizontal load monotonically increasing to failure by Lefas et al. (1990). The experiment included structural walls with constant thickness and a height-to-width ratio varying between 1 and 2. Such walls were considered to represent the critical story elements of a structural wall system with a rectangular cross section. Three levels of constant axial load were adopted in the testing program: 0.0, 0.1 and 0.2 of the uniaxial compressive strength

of the wall cross section that was equal to $0.85f_{cu}bl$ (where b is the thickness and l is the width of the wall, and f_{cu} is the compressive strength of the concrete cube), to represent the amount of axial load at the base of the wall of a single story, a medium-height, and a high-rise building, respectively. It was found that while axial compression reduces both vertical and horizontal displacement, it causes an increase in horizontal load-carrying capacity and secant stiffness. In contrast to what is widely believed, the horizontal web reinforcement did not appear to have a significant effect on shear capacity, as suggested by Wood (1990). The strength and deformation response of the walls were found to be independent of the uniaxial concrete strength characteristics within a range of 30 MPa to 55 MPa. Shear resistance appeared to be associated with the development of triaxial compressive stress conditions within the compression zone near the base of the wall where the flexural moment attained its maximum value. This conclusion was consistent with that proposed by Kotsovos (1985) who demonstrated that the behavior of an element of concrete in a structure can not be described in terms of uniaxial stress-strain characteristics. Such a description not only unnecessarily complicates code provisions for ultimate limit state design but also is not theoretically correct. Design provisions should be based on an understanding of the causes of the observed structural behavior and this can only be achieved by considering the triaxial stress conditions that always develop within a structure under load.

In order to investigate the effect of loading history and repair methods on the structural characteristics of reinforced concrete walls, four identical wall specimens were constructed at a scaled 1:2.4 with constant thickness and a height-to-width ratio equal to 2 and tested by Lefas and Kotsovos (1990). It was found that, while repairing only the damaged regions of the compressive zone was sufficient to fully restore wall strength, the additional use of epoxy resins to heal major flexural and

inclined web cracks led to only a marginal improvement of the structural characteristics such as stiffness and energy dissipation at the serviceability limit state, both being distinctly inferior to those of the original walls under cyclic loading. Such results are in compliance with the concept of the compressive path and demonstrate that, in contrast to widely held views, the compressive zone is the main contribution to shear resistance.

To quantify the true ductility supply and energy absorption potential of RC walls, six RC walls at a 1:2.5 scale were tested under cyclic load by Pilakoutas and Elnashai (1995). Various types of failure were observed, depending mainly on the amount and distribution of shear reinforcement. Shear force was partly transmitted by the concrete in compression and partly by the link reinforcement that enabled shear stress to be resisted through the concrete in the tensile zone. And the strength and deformation characteristics of specimens were not affected significantly by shear reinforcement in excess of the amount required to resist the maximum applied load. Structures designed to sustain extreme forces inelastically should be analyzed based on reduced cracked section stiffness by taking into account ductility considerations. The effect of confinement reinforcement on the ductility was more profound than on the moment capacity. By decomposition of the total deformation in flexural and shear components, it can be demonstrated that the major bulk of the energy dissipation is due to flexure. Net longitudinal extension was expected in flexural RC elements not subjected to significant axial loads due to cyclic loading. Most of the extension took place in the lower part of the wall, where moments exceed the yield capacity. Longitudinal strains on reinforcement near the top of cantilever shear walls were found to be significantly higher than analytically expected. This indicates that an arch-and-tie mechanism was also mobilized for shear resistance.

Tasnimi (2000) tested four identical 1:8 scale wall specimens with constant thickness and a height-to-length ratio equal to 3. These specimens were experimentally subjected to various slow cyclic horizontal loading regimes. The crack patterns and failure modes of all specimens were not fully in compliance with the concept of the compressive force path reported by Lefas and Kotsovos (1990). And this indicates that the capacity of walls is affected by the flexural as well as shears strength of the specimen. The strength and deformational responses of specimens were found to be independent of the cyclic loading sequence.

There has been a great deal of research done on the shear strength of shear walls, deep beams, and corbels. Formulas and recommended rules for design abound in the different building codes and research publications. Siao (1994) attempted to unify the results of all the research efforts to achieve a single consistent approach in the design and analysis of such important structural members. Using formula established for top-loaded deep beams and corbels, the predicted shear capacity of reinforced concrete shear walls showed reasonable good agreement with the experimental results.

2.3 Composite Shear Walls

2.3.1 Composite Members

Steel reinforced concrete (SRC) composite structures used to resist lateral load due to earthquake or wind have been widely explored and developed, with increasing recognition benefits from several different types of composite structures. Wakabayashi (1986) presented that “buildings of composite construction showed good earthquake resistant capacity under the Kanto earthquake (1923) as compared with ordinary reinforced concrete structures”.

In addition, the 1994 Northridge earthquake revealed some unexpected problems regarding the seismic response of the steel moment-resisting frames, especially their connection regions (FEMA, 2000). Cracks were found to occur in the fully-restrained girder-flange-to-column-flange weld and even to propagate into the column flange and web, leading to expensive structural repairs. A series of solutions have been proposed to eliminate or reduce these connection problems directly, such as the removal of backing bars and the use of high quality welding materials (FEMA, 2000). However, a final consensus has not been reached in the earthquake engineering community regarding modifications to the design and construction procedure for moment-resisting frames. An alternative method is to reduce the seismic demands on steel frames by avoiding the use of fully-restrained connections. This philosophy led to the development of the composite steel reinforced concrete wall system, which originates as reinforced concrete shear walls with encased flat steel bars, steel trusses and steel plates (Tall Building Committee A41, 1979), utilizing the high shear stiffness of the reinforced concrete wall to attract most of the lateral shear forces from earthquake motions.

In composite steel-concrete lateral load resisting system, the underlying principle in bringing these distinctly different materials together is to benefit the advantages of both: the inherent stiffness and economy of concrete, the speed of construction of a conventional steel building. The use of composite construction resulted in greater ductility with the load-carrying capacity. The composite system has the desired characteristics of the conventional reinforced concrete system, such as high strength, stiffness, ductility, and fire resistance, and has been found to be cost effective for a range of building heights under certain design conditions.

The Composite Structural Steel and Reinforced Concrete Buildings Provisions

of AISC (1997) and the National Earthquake Hazards Reduction Program (NEHRP, 1997) provide some design provisions for boundary element shear walls.

In order to make clear the structural characteristics of steel profile encased reinforced concrete composite structural systems developed widely as high rise buildings in the half century in Japan, Yamada (1990) studied the steel encased reinforced concrete composite frames. Tests on steel profile encased reinforced concrete composite 3-span, multi-storey rigid frame systems with or without shear wall models were carried out until fracture and compared with ordinary steel or reinforced concrete rigid frames with or without bracing or shear walls. Initial stiffness and ultimate horizontal resistance of these composite systems were between those of the corresponding steel frame or reinforced framework systems. This may enable to present comfortable states for usages in high-rise buildings.

Tupper (1999) studied the reinforced concrete walls with steel boundary elements. Three specimens meeting the requirements of standard reinforced concrete ductile flexural walls were constructed and tested under reversed cyclic loads. One of the composite walls used rectangular hollow structural sections (HSS) as boundary elements which were connected to the wall by welding the transverse bars directly to both HSS elements. The second wall used steel channels connected to the wall with headed studs welded to the channels. These studs overlapped with the transverse reinforcing bars, which had headed ends. The details of these three walls were chosen such that all of the walls had approximately the same flexural capacity. The reversed cyclic response of the three walls showed that each wall had similar hysteretic properties. And the design used for ductile flexural walls was modified to enable comparable design of reinforced concrete walls with steel boundary elements.

Another kind of widely used composite shear wall is steel frame with an infill of reinforced concrete. Although initial design provisions for the composite steel frame-RC infill wall systems have been proposed in NEHRP (1997), this system is rarely used in practice in the United States due to lack of sufficient research. In Japan, design recommendations for earthquake-resistant design of this type of composite system have been implemented (AIJ, 1985). Makino et al. (1980) and Makino (1984) showed that two buildings of this type behaved satisfactorily during the Great Kanto Earthquake of 1923 in Japan, while other buildings were badly damaged.

Chrysostomou (1991) pointed out the importance of composite shear connection between the infilled concrete and steel boundary elements. Without adequate shear connection, the storey shear was primarily resisted by a compression strut in the concrete panel. This compressive strut had finite width and was aligned with the corners of the panel. In contrast, frames with adequate shear connectors were able to resist the shear with a field of diagonal compression in the concrete rather than one single strut.

In the composite steel frame-RC infill system, the steel columns and girders serve as boundary members to resist gravity loads and most of the overturning moment, while the RC infill walls carry most of the shear forces. The RC infill walls had the benefit of increasing the lateral stiffness dramatically, thus avoiding excessive drift and reducing the seismic demands on the steel frames. This provides the opportunity to use partially-restrained (PR) connections in conjunction with lighter steel framing instead of more expensive fully-restrained connections, which are required in the more common steel moment frames (without infill walls). Adequate interface connections, such as headed shear connectors, can be used to transfer shear forces uniformly between the steel frames and infill walls to ensure

effective composite action. This reduces the probability of unfavorable failure mechanisms such as the premature formation and crushing of the compression struts in the RC walls. Experiments conducted by Makino et al. (1980) and Makino (1984) indicated that this system develops a pattern of closely-spaced cracks with small widths. Such cracks may be repaired at a reasonable cost after a moderate earthquake.

Tong (2001) studied the behavior of composite steel frame-reinforced concrete infill wall structure system. This research conducted a cyclic loading test on one two-story, one-bay specimen, which represented the bottom two stories of a six-story prototype structure at approximately one-third scale. Partially-restrained (PR) connections were used to join the wide flange steel columns and beams. The infill was cast using regular concrete. Confined headed studs were used as the interface connectors. The test showed that this system provided sufficient strength to resist the design lateral loads and ample stiffness to control system drift. Adequate redundancy was provided to the system by alternate load paths, such as shear stud-infill interaction and truss action due to steel frame-infill interaction. At the design lateral load level, 80-90% of the lateral shear force was transferred from the steel frame to the infill through the headed studs, and approximately 80% of the overturning moment was carried by the steel columns. The ultimate strength of the specimen was induced by crushing of the concrete in the corners, yielding and fracture of the studs, and formation of plastic hinges in the partial rigid connections. Adequate confining reinforcement was critical to enhance the strength and ductility of the studs in the infill panels. The failure of the studs was controlled by the low cycle fatigue.

Realizing the connection between the composite columns and concrete beam-slab floor as well as the shear-wall edge members and concrete wall are

critical, Shirali (2002) studied the connection design at the interface of the different elements. Ten alternative interface designs, reflecting a one-third scale of the edge region of the first story wall of an 8-story building, have been developed and tested. In a nonlinear FEA, two models, one a nonlinear spring model and the other a truss-like model, have been developed to capture the interface behavior between edge columns and RC wall panels.

Today, the manner of use of steel reinforced concrete walls is based on construction sequence, because the steel frame must be erected before the concrete is placed. The construction of this type of composite system needs a lot of vertical formwork, costly labor work and also causes instability of the steel frames exceeding concrete. In order to make such a shear wall more flexible and applicable and improve construction speed, the concept of “precast” is introduced into this field in current research. Known from the precast structures, the connection must have the ability to transfer the force between adjacent members reliably.

2.4 Connection

The design and construction of connections is the most important consideration in precast concrete structures. Their purpose is to transmit forces between structural members and/or to provide stability and robustness. There are several different ways of achieving a satisfactory connection, e.g. bolting, welding, or grouting, but the method should be simple and must convey unambiguous messages to the site operative.

In the horizontal connections of shear walls, the shear and flexural moment exist simultaneously. The flexural moment at the connection can be readily transferred by connecting vertical steel bars or plates on different parts by several

means, for example, welding, coupler, bolting. Since the aim of the shear wall is to resist the lateral force transferred from other structural members, how to deal with the shear connection is the focus in design of the horizontal connections of shear walls.

Ductile connections used precast large panel systems has been investigated in US PRESSS program. The vertical ductile connectors were designed weaker than wall panels and exhibited some energy dissipation characteristics.

2.4.1 Shear Connection

Shear transfer may be achieved locally using mechanical shear connections. In order to ensure high shear stiffness, the connection is made either by site-welding embedded plates, or by tightly clamping using friction-grip bolts. When gravity action exists on the horizontal connection of precast structure, the friction between two surfaces is an important portion to resist the relative displacement of two parts of the shear wall. This kind of connection has been studied by Soudki (1994) and is out of the scope of present research.

Foerster et al. (1989) studied shear connections for precast panels, two of these connections used continuity reinforcement bars and mechanical shear connectors. Behavior of the specimen suggested that the presence of the mechanical shear connectors enhanced the clamping action and overall ductility of the joint, but one type of connector failed in anchorage of the bars. Only the behavior of the connection under monotonic load was studied in the research.

2.4.2 Diagonal Reinforcement

In order to effectively transfer the shear force between two parts of the wall, two kinds of reinforcement are usually welded to the embedded steel plates. One is vertical reinforcement, the other is diagonal reinforcement.

On the experimental side, work on the use of diagonal reinforcement in RC walls was first studied by Iliya and Bertero (1980) who used grids of diagonal reinforcement in the web of their specimens. They pointed out that the cost of constructing diagonally reinforced walls was higher because the bars must be cut at varying lengths and the placing and anchorage of these bars was more difficult than placing in vertical and horizontal bars. Nevertheless, their tests indicated that the 45° arrangement of the wall reinforcing bars was more effective in resisting the effect of shear reversals, i.e., there was less stiffness deterioration with displacement reversals. In these specimens with the same area of shear reinforcement, in conventionally reinforced specimens' failure was mainly due to diagonal cracking, whereas in the specimen with diagonal web reinforcement the failure was largely flexural.

In order to reduce excessive sliding shear displacements under cyclic reversed loading, particularly at the wall's base immediately above the base beams, two walls with diagonal reinforcement were tested by Paulay et al. (1982). These walls duplicated two other conventionally reinforced walls that had approximately equal flexural and shear strengths. Tests of walls have shown the detrimental effects of excessive sliding shear displacements as well as the marked improvement of response when some diagonal reinforcement crossing the sliding plane was used to reduce displacements and increase the forces resisting sliding shear. The dramatic reduction of the strength, particularly in the second cycles of loading to the same level of ductility was evident in the wall without diagonal reinforcement. The cross-inclined diagonal reinforcement was effective in both tension and

compression and considerably improved seismic response of squat shear walls, although only 30% of the applied shear was resisted by such reinforcement. However, when the diagonal bars yielded due to slip displacements, with cumulative ductility the test unit with diagonal reinforcement dissipated approximately 70% more energy than its counterpart without diagonal reinforcement.

Experimental studies by Fiorato et al. (1983) at the Portland Cement Association included tests of a previously damaged wall that was repaired by removing the web concrete, adding diagonal reinforcement, and placing new concrete. Test results indicated that the deformation capacity of this wall which was susceptible to web crushing was increased by adding the diagonal reinforcement. The observed degradation in stiffness with cycling was also less in the repaired specimen.

Because the available experimental data concerning the behavior of the squat RC walls subjected to high cyclic shear were inconclusive, especially with respect to the sliding shear failure and the role of inclined bars in preventing it, Salonikios et al. (1996, 1998, 1999, 2000, 2002) tested 11 specimens which were reinforced against shear, either conventionally (orthogonal grids of web reinforcement), or with cross-inclined bars. The effect of web and edge reinforcement ratio, axial load level, and quality of construction joints were also investigated. Two sets of specimens were designed: the first consisted of five specimens with an aspect ratio 1.0 (LSW series) and the second set consisted of six specimens with an aspect ratio 1.5 (MSW series). These specimens were designed with respect to the EC8 (1995), which adopted different equations for design against each possible shear failure mode, i.e., diagonal tension (truss mechanism), diagonal compression (crushing of concrete struts), and sliding. LSW2 and LSW3 had the same web reinforcement;

LSW1 was similar to LSW2, but had double the web reinforcement. LSW4 and LSW5 both included bidiagonal reinforcement. LSW5 was similar to LSW2 in every respect, except three bars inclined by 45 degree were added. In order to yield a meaningful comparison of the relative effect of the two types of shear reinforcement, the bidiagonal reinforcement was made in such a way as to achieve an effective reinforcement ratio for bars crossing the potential (horizontal) sliding plane almost identical to that of the vertical web reinforcement in LSW1. LSW4 had the same amount of the bidiagonal reinforcement as LSW5, but the intersection of the 45 degree inclined bars was at the base of the wall. MSW1 to MSW5 were reinforced in a similar way as LSW1 to LSW5. As for MSW6, the base and main part were cast at different times and the specimen reinforcement was connected at the base by lap splicing, without concrete roughening and or additional reinforcement. This allowed studying the effect of a poorly detailed cold joint. All specimens failed in a predominantly flexural mode, characterized by concrete crushing and reinforcement buckling at the confined edges. Pinching of the hysteretic loops was significant in the conventionally reinforced specimens and considerably less in the specimens with diagonal reinforcement.

On the analytical side, Sittipunt and Wood (1995) used finite element method to study the influence of the web reinforcement on the cyclic response of structural walls. In U. S. building codes, the nominal shear strength of slender walls is assumed to increase in proportion to the amount of horizontal web reinforcement. However, many walls subjected to cyclic loading failed due to web crushing after yielding in flexure. So, in Sittipunt's study, six walls with different web arrangements were analyzed. Walls in which only the amount of vertical or horizontal reinforcement was changed exhibited the same overall response. Large average shear strains were calculated at the base of the walls, which led to severely pinched hysteretic curves. These calculations indicated that reasonable increases in

the amount of the horizontal or vertical web reinforcement are not sufficient to improve the cyclic response of walls or to reduce the shear distortion at the base. Walls with diagonal reinforcements exhibited better behavior, lower shear strains in the lower portions of the walls. Through simulation, the authors found that: for the walls with diagonal bars, the applied load versus top deflection curves were more rounded, the shear stiffness did not degrade appreciably with cycling, the shape of the shear distortion curve was improved by decreasing the region of extremely low shear stiffness, and the amplitude of the average shear strain in the lower portion of the walls was reduced. The presence of the diagonal reinforcement did not change the calculated strength of the walls appreciably. This significant improvement in performance may be explained by considering that the diagonal reinforcement runs nearly perpendicular to the cracks that develop in the web of the wall. Therefore, after cracking, most of the force is carried across the cracks by the diagonal reinforcing steel in tension. Under uniaxial vertical loading, the reinforcement displayed well-rounded hysteretic loops and was capable of dissipating energy efficiently. In the walls with only vertical and horizontal web reinforcement, the bars were oriented 30 to 45 degrees to the cracks, and most of the load initially must be carried across the cracks through dowel action. Force transfer through dowel action was characterized by extremely pinched hysteretic curves and stiffness degradation. This phenomenon-pinched effect caused by force transferred by dowel action has been studied by Elliot et al. (1974), Vallenias et al. (1979) and Jimenez-Perez et al. (1987).

Collins et al. (1996) suggested a general methodology for the design of beams and columns against shear. The shear strength in a section defined by an inclined crack is given by the formula:

$$V_n = 4 \cot \theta / [1 + \sqrt{500 \epsilon_1}] (\sqrt{f_c}) b_v d_v + A_v f_v d_v \cot \theta / s \quad (2-1)$$

where θ = the inclination angle of the crack,

ε_1 = the principal tensile strain,

f_c = the concrete's compression strength,

b_v = the effective web width,

d_v = the effective shear depth,

A_v = the area of the shear reinforcement within distance s ,

f_y = the steel yield strength,

s = the spacing of shear reinforcement.

Salonikios et al. (1996, 1998, 1999, 2000, 2002) tested 11 specimens and gave the relevant methodology. During wall deformation, there were three fundamental mechanisms that resisted the imposed deformations. These were the flexural, diagonal shear and sliding shear mechanisms. Although these mechanisms were assumed for simplicity to act independently for the design of the structure, it has been experimentally shown that they strongly interacted with each other. Especially the flexural and sliding shear mechanisms were directly coupled, mainly after heavy flexural cracks. Across the horizontal cracks yield of the sliding shear mechanism was observed because the other resistance mechanisms became less important while cracks propagate and cyclic forces were imposed. The mechanisms that gradually became less important were friction that developed at the crack surface, aggregate interlock, and dowel action.

For inclined shear cracks as shown in Figure 2.4.2-1:

$$\begin{aligned}\sum M_c &= 0 \Rightarrow T_1 d_v + T_d d_v \cot \theta + T_h d_v \cot \theta / 2 + T_l d_v / 2 + 2 T_{dg} \sin \phi d / 2 \\ &= V_u d_v \cot \theta + M_u - N d_v / 2 \\ &\Rightarrow T_1 + T_d \cot \theta / 2 + T_l / 2 + 2 T_{dg} \sin \phi / 2 \\ &= V_u \cot \theta + M_u / d_v - N_u / 2\end{aligned}\quad (2-2)$$

$$\sum F_x = 0 \Rightarrow V_u = T_h + T_{fr} \sin \theta + T_{cf} + T_d + T_{dg} \cos \phi \quad (2-3)$$

$$\sum F_y = 0 \Rightarrow N_u = T_{fr} \cos \theta + T_c - T_1 - T_l - T_{dg} \sin \phi \quad (2-4)$$

where T_1 = the tensile strength of the flexural reinforcement,
 T_d = the dowel action strength of the horizontal web reinforcement,
 T_h = the tensile strength of the horizontal web reinforcement and dowel
 action of vertical web reinforcement,
 T_l = the tensile strength of vertical web reinforcement and the dowel
 action of the horizontal web reinforcement,
 T_{dg} = the tensile strength of bidiagonal reinforcement,
 T_{fr} = the aggregate interlock and friction resistance force, along the crack
 in the web,
 T_{cfr} = the aggregate interlock and friction resistance force at the edge
 column,
 T_c = the compressive force at the edge column,
 ϕ = diagonal reinforcement inclined angle,
 φ = angle between tensile strength and horizontal line.

Considering the influence of reduced axial force and aspect ratio,

$$T_{cfr} = 2.15v(0.75M_{\max} / d_v + N_u / 2) / \alpha_s \quad (2-5)$$

$$T_d = 0.385(f_u A_{sfl}) \alpha_s \quad (2-6)$$

where $v = N_u / f_c A_c$,

$$\alpha_s = M_u / (V_u l_w),$$

f_u = the tensile strength of reinforcement,

A_{sfl} = the area of edge column longitudinal reinforcement.

In specimens with bidiagonal reinforcement:

$$\sum F_x = 0 \Rightarrow V_{usl} = T_{cfr} + [1 + 0.1l_i / (0.8l_w)](0.27T_d + 2T_{dg} \cos \phi) \alpha_s \quad (2-7)$$

where $T_{dg} = f_y A_{sd}$,

A_{sd} = the area of the diagonal reinforcement in direction,

l_i = the distance between the inclined bars' center axes, at the base cross the section,

l_w = the length of the wall.

Fig.2.4.2-1 External acting forces and internal reactions at shear wall inclined crack in direction θ

From the above experimental and analytical experience, the effect of diagonal reinforcement on reducing shear deformation, stiffness degradation under cyclic loads, and increasing the shear strength is apparent. So the diagonal reinforcement was selected in the present study to control the shear deformation (relative horizontal displacement of two panels) of the specimens.

2.5 Strut-and-tie Model

Rational design of reinforced concrete structures must be based on the mechanical properties of the materials. Concrete has high strength in compression but very low strength in tension. When concrete is used in a structure to carry loads,

the tensile regions are expected to crack and, therefore, must be reinforced by materials of high tensile strength, such as steel. This concept of utilizing concrete struts to resist compression and reinforcing ties to carry tension gave rise to the term “strut-and-tie model”. In this model, the concrete compression struts and the steel tension ties form a truss that is capable of resisting the applied loads. As such, this concept is also called the “truss model”. This method implies that the structure is designed according to the lower bound theorem of plasticity. Because concrete permits only limited plastic deformations, the internal structural system (the strut-and-tie model) has to be chosen in a way that the deformation limit (capacity of rotation) is not exceeded at any point before the assumed state of stress is reached in the rest of the structure.

2.5.1 History of Strut-and-Tie models

Ritter (1899) and Morsch (1902) conceived the truss model concept, the angle of the concrete struts was conveniently assumed to be 45° . After Mohr’s compatibility condition was established, however, it was realized that 45° inclination of struts represents the equal strain condition of the longitudinal and transverse steel.

Shear design procedures for reinforced concrete that, like tension field theory, determine the angle θ by considering the deformations of the transverse reinforcement, the longitudinal reinforcement, and the diagonally stressed concrete have become known as compression field theory (CFT). Kupfer (1964) and Baumann (1972) presented approaches for determining the angle θ assuming that the cracked concrete and the reinforcement were linearly elastic. Based on the theory of plasticity, Nielson (1967) and Lampert and Thurlimann (1968) used the membrane elements to analyze the RC structures. They first established the two-dimensional

equilibrium condition of reinforced concrete membrane elements subjected to shear and normal stresses. Elfegren (1972) developed a truss model which was based on the equilibrium condition that assumed both the longitudinal and the transverse steel yield at the ultimate load stage. The term “compression stress field” conveys the assumption that the direction of the concrete struts coincides with the direction of the post-cracking principal compression stress. Methods for determining θ applicable over the full loading range were developed by Mitchell and Collins (1974) for members in torsion, and were applied to shear design by Collins (1978).

Prior to 1972, the truss model severely overestimated the shear and torsion strengths when the failure stress of the concrete struts was assumed to be the uniaxial compression strength of standard concrete cylinders. Robinson and Demorieux (1968, 1972) observed that a reinforced concrete element under shear is actually subjected to a two-dimensional stress condition. The strength in the principal compression direction was found to be softened by the principal tension in the perpendicular direction. Vecchio and Collins (1981) proposed the first softened stress strain curve of concrete in compression. This curve incorporated a softened coefficient, which is a function of the principal tensile strain. The softened stress-strain curve of concrete was used in combination with Mohr’s stress and strain circle to predict the behavior of reinforced concrete elements. Collins used the “compression field” to reflect the assumption that the direction of the concrete struts coincides with the direction of the post-cracking principal compression. In order to derive the “contribution of concrete”, Vecchio and Collins (1986) proposed the “modified compression field theory (MCFT)”.

On the basis of equilibrium, compatibility and constitutive laws of material, “softened truss model” was developed by Hsu (1988) and Pang and Hsu (1996). Hsu (1996) proposed a unified nomenclature that covered all four basic actions

(bending, axial load, shear and torsion), which was based on the angle of inclination of the concrete struts, the scope of applications, and three fundamental principles of the mechanics of materials (stress equilibrium, strain compatibility and constitutive laws of materials). Six models, including a strut-and-tie model, a Bernoulli compatibility truss model, an equilibrium (plasticity) truss model, a Mohr's compatibility truss model, a softened truss model, and a fixed-angle softened-truss model were provided in this study. The constitutive laws of concrete required in the rotating angle softened truss model have been formulated as shown in Fig. 2.5.1-1 by Belarbi and Hsu (1995) and Pang and Hsu (1996). The average stress-strain curve of concrete in compression is given as:

$$\sigma_2 = \zeta f'_c \left[2 \left(\frac{\varepsilon_2}{\zeta \varepsilon_0} \right) - \left(\frac{\varepsilon_2}{\zeta \varepsilon_0} \right)^2 \right] \quad \varepsilon_2 / \zeta \varepsilon_0 \leq 1 \quad (2-8)$$

$$\sigma_2 = \zeta f'_c \left[1 - \left(\frac{\frac{\varepsilon_2}{\zeta \varepsilon_0} - 1}{\frac{2}{\zeta} - 1} \right)^2 \right] \quad \varepsilon_2 / \zeta \varepsilon_0 \geq 1 \quad (2-9)$$

$$\zeta = \frac{0.9}{\sqrt{1 + 400 \varepsilon_1}} \quad (2-10)$$

where f'_c = maximum compressive strength of standard 6×12-in concrete cylinder,

ε_0 = concrete strain at maximum compressive strength taken as 0.002,

ζ = softening coefficient.

Fig.2.5.1-1 Compressive stress-strain curve of concrete

Based on the strut-and-tie concept, equilibrium, compatibility and constitutive laws of cracked reinforced concrete, Hwang and Lee (1999, 2000) and Hwang et al. (2000, 2001) proposed a softened strut-and-tie model to predict the strength of the beam-column joints, squat walls and deep beam; and the effect of the boundary elements, cyclic loading and vertical loads on the wall and shear strength also were studied. This model was used to predict the shear strength of deep beams and squat walls. Each analytical result agreed well with the corresponding experimental result in the study. After the development of the first cracking pattern in the wall, the steel bars were subjected to tension and the concrete acted as compressive struts, thus forming a strut-and-tie action. Three strut-and-tie load paths were proposed to model the force transfer within the squat wall, and they are diagonal, horizontal, and vertical mechanism as shown in Fig.2.5.2-2.

Fig.2.5.2-2 Wall shear resisting mechanisms: (a) diagonal; (b) horizontal; (c) vertical

The diagonal mechanism is a single diagonal compression strut whose angle of inclination θ is defined as:

$$\theta = \tan^{-1} \left(\frac{H}{l} \right) \quad (2-11)$$

It is assumed that the direction of the diagonal concrete strut coincided with the direction of the principal compressive stress of the concrete. And the effective area of the diagonal strut A_{str} is defined as:

$$A_{str} = a_s \times b_s \quad (2-12)$$

where a_s = depth of the diagonal strut,

b_s = width of the diagonal strut that can be taken as the width of the wall

web t_w .

The depth of the diagonal strut depends on its end condition provided by the compression zone at the base of the wall. It is intuitively assumed that $a_s = a_w$, where a_w is depth of the compression zone at the base of the wall. For simplicity, a_w can be approximated with Paulay and Priestley's (1992) equation for the depth of the flexural compression zone of an elastic column.

$$a_w = \left(0.25 + 0.85 \frac{N}{A_w f'_c} \right) l_w \quad (2-13)$$

where A_w = net area of the concrete section bounded by the web thickness t_w

and the length of the section in the direction of the shear force l_w ,

f'_c = compressive strength of the concrete.

N = vertical force acting on the wall.

The horizontal mechanism included one horizontal tie and two flat struts as shown in Fig.2.5.2-2. The horizontal tie was made up of horizontal shear reinforcement. When computing the cross sectional area of the horizontal tie, it is assumed that the horizontal shear reinforcement within the center half of the wall is fully effective, and the other horizontal steel is included as 50% effective. The vertical mechanism is composed of one vertical and two steep struts. The vertical tie includes only the vertical shear reinforcement within the wall web and excludes the vertical reinforcement of the boundary elements. For a wall without boundary elements, the vertical shear reinforcement within the central portion of $0.8l_w$ is considered effective to constitute the vertical tie. For diagonal compression failure,

the shear strength of the squat wall is defined as the concrete compressive stress on the nodal zone as the concrete reaches its capacity. The boundary of the nodal zone coincides with the boundary diagonal strut, but the concrete bearing force to be examined is the summation of the compression from the diagonal, flat and steep struts.

Because the diagonal compression is mainly transferred in D-direction shown in Fig.2.5.2-2, the maximum compressive stress acting on the nodal zone:

$$\sigma_{d,max} = \frac{1}{A_{str}} \left\{ D - \frac{\cos\left(\theta - \tan^{-1}\left(\frac{H}{2l}\right)\right)}{\cos\left(\tan^{-1}\left(\frac{H}{2l}\right)\right)} F_h - \frac{\cos\left(\tan^{-1}\left(\frac{2H}{l}\right) - \theta\right)}{\sin\left(\tan^{-1}\left(\frac{2H}{l}\right)\right)} F_v \right\} \quad (2-14)$$

Where D is depth of diagonal strut,

F_h is sum of forces in horizontal reinforcement;

F_v is sum of force in vertical reinforcement in central $0.8l_w$

2.5.2 B Region and D Regions

The body of a reinforced concrete structure can be visualized as having two typical regions: B regions and the D regions.

- B regions are referred to the main regions in which the Bernoulli hypothesis of plane strain distributions is assumed valid. That is, the stresses and strains in the main regions are governed by simple equilibrium compatibility conditions. The internal state of stress of these regions is easily derived from the sectional forces (bending and torsional moments, shear and axial forces). It is logical to apply truss model to the main regions using a regular angle of concrete struts.
- D regions are local regions where the strain distribution is significantly nonlinear and compatibility conditions are difficult to apply, e.g., near

concentrated loads, corners, bends, openings and other discontinuities. The main types of discontinuity are geometric discontinuity and load discontinuity as shown in Fig.2.5.2-1 and Fig.2.5.2-2, respectively. In these regions the stresses and strains are so disturbed and irregular that they are not easy to get a mathematical solution. In the design of local regions, the stresses are determined by equilibrium conditions alone, while the strain compatibility conditions are neglected. So, the strut-and-tie model with irregular angles is applied to the local regions. The local region is treated as an isolated free body subjected to external boundary forces. Boundary forces are calculated from the four actions diagrams including bending moment and torsion, shear and axial force at the boundary sections.

Fig.2.5.2-1 Geometric Discontinuity (ACI 318-02, Appendix)

Fig.2.5.2-2 Load Discontinuity (ACI 318-02, Appendix)

2.5.3 Design Procedure of Strut-And-Tie Model

In designing a reinforced concrete structure, strut-and-tie models can be used to directly design members and detail. The design procedure is shown in Fig.2.5.3-1 keeping in mind then following requirements:

- 1) Equilibrium must be maintained
- 2) Tension in concrete is neglected
- 3) Forces in struts and ties are uniaxial
- 4) External forces apply at nodes
- 5) Prestressing is treated as a load
- 6) Detailing for adequate anchorage

Design Procedure and a flowchart is shown in Fig.2.5.3-2:

- 1) Determine loads and reactions and estimate dimensions and member sizes. Considering design requirements of clients and actual surrounding natural conditions, structural system is selected. The loads and reactions will be gotten through calculation. According to requirements of design codes, dimensions and member sizes can be estimated.
- 2) Define B- and D-regions and isolate each D-region. In using the strut-and-tie model approach, it is helpful and informative to first subdivide the structure into its B- and D-regions. The truss model and the design procedure for the B-regions are then readily available and only the strut-and-tie models for the D-regions remain to be developed and added. In defining B- and D- regions, the definition of geometric discontinuity and loading discontinuity is helpful.
- 3) Compute resultant forces on each D-region boundary. These resultant forces include resultant forces from adjacent B-regions and reactions from supports.
- 4) Select a truss model to transfer the resultant forces across the D-region. The axes of the struts and tie, respectively, are chosen to approximately coincide with the axes of the compression and tension fields. The forces in the struts and ties are computed.
- 5) The effective widths of the struts and nodal zones are determined considering the forces from step 4 and the effective concrete strengths and reinforcement is provided for the tie considering the steel strength. The reinforcement should be anchored in the nodal zones.

Fig.2.5.3-1 Design Flow Chart of Reinforced Concrete Structure

a) B- and D-Regions (ACI 318-02, Appendix)

b) Strut-and-Tie Model for Entire Beam (ACI 318-02, Appendix)

c) Detail at End of Beam

Fig.2.5.3-2 Strut-and-Tie Model for Simple Span Beam (ACI 318-02, Appendix)

2.5.4 Method for Developing Strut-and-Tie Models

A. Elastic Analysis Approach

The struts and ties in strut-and-tie models condense the real stress field by resultant straight lines and concentrate their curvature in nodes, which means that developing the model is much more simplified if the elastic stresses and principal stress directions are available. Such an elastic analysis is readily facilitated by the wide variety of computer programs available today. The direction of struts can be taken in accordance with the mean direction of principal compression stress or the more important struts and ties can be located at the center of gravity of the corresponding stress diagrams.

Fig.2.5.4-1 Strut-and-Tie Based on Elastic Stresses (Schlaich, 1987)

Fig.2.5.4-2 Strut-and-Tie Based on Elastic Stresses (Schlaich, 1987)

B. The Load Path Method

However, even if no elastic analysis is available and there is no time to prepare one, it is easy to develop strut-and-tie models using so-called “load paths.” The comparison between load path and strut-and-tie model is shown in Fig.2.5.4-1.

First, it must be ensure that the outer equilibrium of the D-region is satisfied by determining all the loads and reactions (support forces) acting on it.

The stress diagram is subdivided in such a way, that the loads on one side of the structure find their counterpart on the other, considering that the load paths connecting the opposite sides will not cross each other. The load paths begin and end at the center of the gravity of the corresponding stress diagrams and have the direction of the applied loads or reactions. They tend to take the shortest possible streamlined way in between. Curvatures concentrate near stress concentrations (support reactions or singular loads)

After plotting all load paths with smooth curves and replacing them by polygons, further struts and ties must be added for transverse equilibrium acting between the nodes, including those of the U-turn. While doing so, the ties be arranged with proper consideration of practicality of the reinforcement layout (generally parallel the concrete surface) and of crack distribution requirements.

A very powerful means of developing new strut-and-tie model for complicated cases is the combination of an elastic finite element method analysis with the load path method.

Fig.2.5.4-1 Strut-and-Tie Model Based on Load Paths (Schlaich, 1987)

2.5.5 Model Optimization

For a structure more than one strut-and-tie models can be developed. Doubts could arise as to whether the correct model has been chosen out of several possible ones. In selecting the model, it is helpful to realize that loads try to use the path with the least forces and deformations. Since reinforced ties are much more deformable than concrete struts, the model with the least and shortest ties is the best.

The simple criterion for optimizing a model may be formulated as follows:

$$\sum F_i l_i \varepsilon_{mi} = Minimum$$

Where

F_i = force in strut or tie i

l_i = length of the member i

ε_{mi} = mean strain of member i

This equation is derived from the principle of minimum strain energy for linear elastic behavior of the struts and ties after cracking. The contribution of the concrete struts can generally be omitted because the strains of the struts are usually much smaller than those of the steel ties.

2.5.6 Dimensioning the Struts, Ties and Nodes

Dimensioning not only means sizing and reinforcing the individual struts and ties for the forces they carry, but also ensuring the load transfer between them by checking the node regions.

There are basically three types of struts and ties to be dimensioned:

C_c : Concrete struts in compression

T_c : Concrete ties in tension without reinforcement

T_s : Ties in tension with reinforcement

A. Struts and Ties

The struts in the model are resultants of the stress fields. Since by definition the curvatures or deviations of the forces are concentrated in the nodes, the struts are straight.

Compression struts serve as the compression chord of the truss mechanism

which resists moment and act as the diagonal struts which transfer shear to the supports. Diagonal struts are generally oriented parallel to the expected axis of cracking.

Three typical configurations of compression fields:

Fig.2.5.6-1 Typical Configurations of Compression Field (Wu Hui 2005)

B. Nodes

Nodes are derived as the intersection points of the three or more straight struts or ties, which themselves represent either straight or curved stress fields or reinforcing bars or tendons. A node implies an abrupt change of direction of forces.

There are essentially four types of nodes depending on the combination of struts C and ties T as shown in Fig.2.5.6-2, which can be idealized by three kinds of nodal zones shown in Fig.2.5.6-3.

Fig.2.5.6-2 Type of Nodes (Schlaich, 1987)

Fig.2.5.6-3 Idealized Forces at Nodal Zone (Schlaich, 1987)

2.5.7 Feasible Inclined Angle θ

The maximum length-to-depth ratio of a D-region would be approximately two. Thus, the smallest angle between the strut and tie in a D-region is $\arctan 2 = 26.5^\circ$. The feasible inclined angle θ is different in different codes:

- Swiss Code: $0.5 \leq \cot \theta \leq 2.0$ ($\theta = 26^\circ$ to 64°)
- European Code: $3/5 \leq \cot \theta \leq 5/3$ ($\theta = 31^\circ$ to 59°)
- Collin's & Mitchells:

$$\theta_{\min} = 10 + 110(V_u / [\phi_c' b_w j d]) \text{ deg}$$

$$\theta_{\min} = 90 - \theta_{\min} \text{ deg}$$
- ACI 2002: $\theta_{\min} = 25^\circ$ ($\theta = 25^\circ$ to 65°)
- If small θ is assumed in the truss model, the compression strength of the inclined strut is decreased.

2.5.8 Limiting Compressive Stress in Strut

As mentioned previously, the limiting stresses for truss elements in a strut-and-tie model is different, as shown in Fig.2.5.9-1, different codes give different requirements on these stresses as following:

A. AASHTO LRFD 5.6.3.3.3

$$f_{cu} = \frac{f_c'}{0.8 + 170\varepsilon_1} \leq 0.85f_c'$$

Where:

$$\varepsilon_1 = (\varepsilon_2 + 0.002) \cot^2 \theta_m$$

f_{cu} = the limiting compressive stress

θ_m = the smallest angle between the compressive strut and adjoining tension ties (deg)

ε_2 = the tensile strain in the concrete in the direction of the tension tie (in/in)

- Strength of compressive strut AASHTO LRFD 5.6.3.3.3

$$P_r = F P_n \quad (\text{LRFD 5.6.3.2.1})$$

$$P_n = f_{cu} A_{cs} \quad (\text{LRFD 5.6.3.3.1-1})$$

Where:

$F = 0.7$ for compression in strut-and-tie models (LRFD 5.5.4.2.1)

A_{cs} = effective cross-sectional area of strut (LRFD 5.6.3.3.2)

B. Simplified Values for Limiting Compressive Stress in Strut, f_{cu} (Schlaich et al. 1987)

- For an undisturbed and uniaxial state of compressive stress:

$$f_{cu} = 0.85 f'_c$$

- If tensile strains in the cross direction or transverse tensile reinforcement may cause cracking parallel to the strut with normal crack width; this applies also to node regions where tension steel bars are anchored or crossing.

$$f_{cu} = 0.8(0.85 f'_c) = 0.68 f'_c$$

- As above for skew cracking or skew reinforcement

$$f_{cu} = 0.6(0.85 f'_c) = 0.51 f'_c$$

- For skew cracks with extraordinary crack width-such as cracks must be expected if modeling of the struts departs significantly from the theory of elasticity's flow of internal forces:

$$f_{cu} = 0.4(0.85 f'_c) = 0.34 f'_c$$

C. ACI 2002 STM Model

Design of struts, ties and nodal zones shall be based on:

$$\phi F \geq F_u$$

The nominal compressive strength of a strut without longitudinal reinforcement:

$$F_{ns} = f_{cu} A_c$$

The effective compressive strength of the concrete in a strut is:

$$f_{cu} = 0.85 \beta_s f'_c$$

- The strength of a longitudinally reinforced strut is:

$$F_{ns} = f_{cu} A_c + A'_s f'_s$$

- The nominal strength of a tie shall be taken as:

$$F_{nt} = A_{st} f_y + A_{ps} (f_{se} + \Delta A_{st} f_p)$$

- The nominal compressive strength of a nodal zone shall be:

$$F_m = f_{cu} A_u$$

Fig.2.5.9-1 Limiting Stress for Truss Elements (ACI 318-02, Appendix A)

2.5.9 Summary on Strut-and-Tie Model

- The strut-and-tie model approach has evolved as one of the most useful design methods for shear critical structures and for other disturbed regions in concrete structures.
- The strut-and-tie model formulation that requires the least volume of steel will be the solution that best models the behavior of a concrete member.
- This approach holds great promise for design offices which could develop or obtain standard strut-and-tie models for certain commonly encountered

situations

- Standard reinforcement details based on a strut-and-tie model could be developed for common situations.
- The strut-and-tie model then could be reviewed and revised if any parameter changes.

2.6 Summary

In this chapter, background and significance of present research is addressed. Past research on performance of shear walls and steel reinforced shear walls has been done including crack patterns, failure modes, flexural and shear deformations, horizontal and vertical dilations, pitching effect, stiffness, strength, and energy dissipation. In these studies, main influential factors including strength of concrete, ratio of flexural and shear reinforcement, layout of reinforcement, aspect ratio, gravity and loading history, have been investigated.

As valuable tools in explaining internal forces of concrete members under shear, strut-and-tie models were applied to analyze behavior of specimens in this research.

CHAPTER 3

EXPERIMENT DESIGN

In order to investigate seismic performance of precast shear wall with horizontal connection and study the effect of parameters: horizontal connection, number of shear connectors and configuration of steel sections, six two-storey reinforced concrete model walls were tested to failure under reversed cyclic lateral load. Two monolithic shear walls acting as prototypes and four precast shear walls with horizontal connection serving as counterparts were tested under cyclic load to failure.

Capacity of energy dissipation of shear walls was mainly controlled by plastic deformation of reinforcement before shear failure. And experiment by Cardenas and Magura (1973) indicated that, under monotonic load, larger ductility could be attained with concentrated reinforcement at the ends of the walls rather uniformly distributed reinforcement. Steel sections were used in these specimens serving as concentrated flexural reinforcement at boundaries of walls, namely, channels were embedded in specimen W1, W2 and W3, whereas, I-beams were utilized in specimen W4, W5 and W6.

Due to concentrated vertical reinforcement placed at two the ends of the walls and the low aspect ratio of these specimens, $\alpha = h_w/l_w = 1.41$, these specimens exhibit possible sliding shear failure under cyclic load. Salonikios et al (1996, 1998, 1999, 2000, 2002) verified that diagonal reinforcement is capable of preventing this failure mode. In the present study, diagonal reinforcement with 45 deg inclined to horizontal was used in lower part of the walls with height equal to $0.22h_w$ from the base.

In precast walls, the horizontal connection consists of shear connectors and flexural connectors.

The effect of confining reinforcement on ductility was more profound than on the moment capacity, which was supported by experimental evidence by Pilakoutas and Elnashai (1995). The precast walls are popular in moderate and low seismic regions, and low ductility is required for these walls. Confinement was not designed in this research.

To get a simple and clear flow of internal forces, a rectangular cross section was selected for all walls. No axial load was applied to the specimens, because it is usually small in structural walls and generally has a favorable effect on their behavior, suggested by Park and Paulay (1975).

The data acquisition system consisted of a series of internal control and recording channels and channels for monitoring data from external instruments-LVDT (linear variable displacement transducers). And a series of strain gauges at various positions along the reinforcing bars and steel sections at specific positions were used to record the strain history during cyclic loading.

The foregoing system of measurements made it possible to estimate the flexural, shear, and sliding components of the wall deformation. Based on strain gauges and cracking patterns of these walls, strut-and-tie models at different stages were developed to analyze performance of these specimens.

3.1 Design of Experiment

3.1.1 General Description of Wall Specimens

Total of six walls specimens were tested to failure in the present research. The dimensions of the walls were 1800mm wide, 2300mm high and 140mm thick and the aspect ratio is $\alpha = h_w/l_w = 1.41$. Two Specimens are monolithic walls, W1 and W4, serving as prototypes, whereas four precast walls, namely, W2 and W3, W5 and W6, are counterparts of W1 and W4, respectively. Channels were laid at two ends of wall W1, W2 and W3, acting as concentrated flexural reinforcement,

whereas I-beams were used in wall W4, W5 and W6. The concrete cover of horizontal reinforcement which was placed outside of vertical bars was 20mm at the wall end sides and 15mm at wall faces.

The concrete used in all specimens is $f_{cu} = 40$ MPa and had 12mm maximum size aggregate. The vertical reinforcement in the walls was of mild yield steel bars, as called R bars, with design yield strength 250MPa conforming to BS4461. And the horizontal reinforcement was of high yield steel bars, called T bars, with design yield strength 460MPa BS4469. And the steel sections, including channels and I-Beams were Grade 43. Same reinforcement including size and layout was used in the six walls except that the vertical reinforcement was cut at the horizontal connection in precast specimens.

In the horizontal connections, all bolts were Grade 8.8, and diameter of hole for diameter equal to 12mm bolts is 13mm, and 22mm for diameter equal to 20mm bolts. The diagonal bars were T bars which welded on steel plate with angle of 25 deg inclined to horizon. Size of steel plates used in specimen W2 and W5 was $400 \times 200 \times 10$ mm and $220 \times 200 \times 10$ mm in specimen W3 and W6.

All specimens were cast vertically in plant. The monolithic ones, W1 and W4, were cast in one time. But, each of the specimens W2, W3, W5 and 6W was divided into two segments, and the two segments were cast separately. Upon seven days of curing, the two segments of each specimen were joined together with bolts, nuts and concrete with same strength of wall panel.

The reversed cyclic lateral load was applied by a testing rig. On base beam, there are fourteen PVC tubes with diameter 52mm evenly distributed along the horizon in two lines. The base beam is firmly fixed on the strong floor of laboratory with steel rods going through these PVC tubes with nuts. A horizontal beam of the testing rig was connected to the top beam of the specimens. The dimensions of the top beam are shown in Fig. 3.1.3-8 and it is a model of the six specimens. There are four 32mm PVC tubes passing through the top beam, which were used as a duct for

loading bars. The loading bars are connected to horizontal loading beam of testing rig by nuts, which work in pulling stages in the loading cycles. The loading conditions and boundary conditions are same for all specimens.

3.1.2 Material Properties

A. Concrete

All specimens were cast vertically indoor with a standard mix. Concrete was supplied by external commercial ready-mix supplier in Singapore. The characteristic strength f_{cu} is 40N/mm² with 100±25mm slump in accordance with BS5328. The maximum size of aggregate is equal to 12mm in order to ensure good compaction of concrete in the specimen walls. A total of eighteen 150×150×150mm cubes and eighteen 150×150×300mm cylinders were cast of each batch. Three cubes and three cylinders were cured for each specimen under identical conditions. Cubes and cylinders were tested at the same day of testing specimen itself and the average of results was taken as the compressive strength as shown in Table 3.1.2-1.

Table 3.1.2-1 Average Concrete Compressive Strength at The Day of Testing

Specimens	Days	Average f'_c (MPa)	Average f_c (MPa)
W1	60	43.51	35.58
W2	70	44.12	33.53
W3	81	44.78	34.04
W4	87	45.35	35.45
W5	94	43.98	34.02
W6	104	44.65	33.24

B. Reinforcement and steel sections

Five samples of each kind of reinforcing bar were tested in tension. In this research, reinforcing bar, T10, was used as horizontal shear reinforcement. The

stress versus strain relationship is shown in Fig.3.1.2-1. Stress vs strain curve of vertical reinforcing bar, R10, is shown in Fig.3.1.2-2. Coupons of web of UB (I-Beam) and Channel shown in Fig.3.1.2-3 were test and stress against strain curve was plotted in Fig.3.1.2-4 and Fig.3.1.2-5. The properties of the reinforcement and steel sections including area of cross section, elasticity modulus, yield strength and ultimate strength are listed in Table.3.1.2-2 .

Fig. 3.1.2-1 Stress Versus Strain Relationship of T10 Bar

Fig. 3.1.2-2 Stress Versus Strain Relationship of R10 Bar

FFig. 3.1.2-3 Coupon design of I-Beam and Channel

Fig. 3.1.2-4 Stress Versus Strain Relationship of Channel

Fig. 3.1.2-5 Stress Versus Strain Relationship of I-Beam

Table 3.1.2-2 Properties of Reinforcement and Steel Sections

Types	R10	R13	T10	T20	Channel	I beam
Area of cross section A_s (mm ²)	78.5	132.67	78.5	314	856	2050
Modulus of Elasticity E_s (MPa)	192721.33	193237.58	185553.79	202120.82	192460.35	177279.69
Yield strength f_y (MPa)	384.63	402.3	485.91	502	307.74	322.16
Yield strain($\mu\epsilon$)	2200	2200	2700	2900	1800	2000
Ultimate strength f_u (MPa)	544.5	502.6	603.68	626.4	470.97	487.78

3.1.3 Design of Specimens

A. Design of specimen W1

The reinforcement layout of specimen W1 is shown in Fig.3.1.2-2. Due to the limit of capacity of testing rig, reinforcement R10 are selected as vertical reinforcement for its lower yield strength and ultimate strength comparing with T10 bar. All the horizontal reinforcement is T10 bars which are welded on the channels at ends of the wall. Every channels embedded in base beam is weld with two bars with diameter equal to 32mm to ensure anchorage of the channels. For specimens with embedded I-Beams, same method is applied to reach requirement of anchorage in base beam.

The initial specimen design is based material properties provided by BS 4449, in which the yield strength of hot rolled mild steel, R10, is 259MPa, and that of high yield steel, T10, is 460MPa. The yield strength of channel is 275MPa. Compression strength of concrete cylinder is 30MPa. According to ACI 318-02, strain in reinforcement and concrete is directly proportional to the distance from the neutral axis. And the assumed maximum usable strain at extreme concrete compression fiber is equal to 0.003.

Try the depth of neutral axis, $c=170\text{mm}$. Therefore, two channels and 16 R10 bars are in tension, 1 channel and 2 R10 bars are in compression.

$$\alpha = 0.85c = 144.5\text{mm}$$

$$C_c = 0.85 \times 144.5 \times 140 \times 30 = 516\text{kN}$$

$$C_s = 2 \times 78.5 \times 250 + 856 \times 275 = 275\text{kN}$$

$$C = C_s + C_c = 791\text{kN}$$

$$T_s = 16 \times 78.5 \times 250 + 2 \times 856 \times 275 = 785\text{kN}$$

For $785\text{kN} \approx 791\text{kN}$, so $T=C$.

$$\begin{aligned} M &= T_{s-\text{channels}} \times (d_1 - \alpha/2) + T_{s-\text{bars}} \times (d_2 - \alpha/2) \\ &= 470.8 \times (1800 - 101 - 144.5/2) + 314 \times (1800 - 700 - 144.5/2) \\ &= 1089\text{kNm} \end{aligned}$$

Where d_1, d_2 are the distances from centroid of channels and that of bars in tension to the extreme compression fiber, respectively.

Therefore, the horizontal force loading at the top beam will be:

$$F = M / h = 1089 / 2.54 = 429\text{kN}$$

Where h is height of wall, measuring from the wall base to horizontal central line of the top beam.

As shown in Fig.3.1.3-1. T10 bars are used as horizontal reinforcement with spacing 200mm. According to ACI 318-02,

$$V_s = \frac{A_v f_y d}{s_2} = \frac{A_v f_y \times 0.8 l_w}{s_2} = \frac{78.5 \times 2 \times 460 \times 0.8 \times 1800}{200} = 520\text{kN}$$

$$\phi_o = \frac{520}{429} = 1.21$$

Due to special layout of vertical reinforcement and low aspect ratio, the walls may fail in sliding shear near the base under reversed cyclic load. In order to prevent this kind of failure, four diagonal bars in each direction was placed in $0.22 h_w$ (550mm) height from the base, as shown in Fig.3.1.3-1. All diagonal bars both under tension and under compression contribute to resist the shear force at the wall base.

$$V_{dia} = A_{dia} f_y \sin 45^\circ = 4 \times 2 \times 78.5 \times 460 \times 0.707 / 1000 = 204\text{kN}$$

B. Design of specimen W2 and W3

The specimen W2 and W3 are precast counterparts of monolithic specimen W1. The horizontal reinforcement in W2 and W3 are identical with that in W1. And the amount of channels and vertical bars used in W2 and W3 is same to that in W1 except they are cut into two.

In each horizontal section of wall under loading point, flexural and shear force exist simultaneously. In present study, only the channels in upper and lower wall panels are connected by a group of bolts with steel plates, which form mechanism to resist the flexural moment at the horizontal connection level. Also, shear connectors embedded in upper and lower wall panels, which are composed of diagonal bars and steel plates and positioned at center of the horizontal connection, are connected by bolts to withstand the shear force and inhibit sliding along the horizontal connection.

The dimensions and horizontal and vertical reinforcement of W2 are identical with those of W3. The difference between specimen W2 and W3 is different of number of shear connectors. In specimens W2 there is only one shear connector but in specimen W3.

In design of connection, in order to get a clear load transfer in the horizontal connection, the flexural moment and shear force are designed separately and resisted independently. The channels at two ends of the walls were used to take flexural moment and shear connectors at center of the connection to resistance the horizontal shear force.

a. Design flexural connectors of specimen W2 and W3

Assuming channels only resist tension and compression force at ends of the walls which form a moment-resistant mechanism. Grade 8.8 bolts with diameter equal 12mm are applied to connect upper and lower channels as shown in Fig.3.1.2-

2, Fig.3.1.2-3 and Fig.3.1.2-9. And diameter of bolts hole is 13mm and web thickness of the channel is 5.1mm.

$$\begin{aligned} \text{So, the effective area of channels} &= \text{area of channels} - \text{area of hole} \\ &= 856 \times 2 - 13 \times 4 \times 5.1 = 1446.8 \text{ mm}^2 \end{aligned}$$

The yield strength of channel of Grade 43 is 275MPa.

Therefore, the strength of flexural connector is,

$$1446.8 \times 275 = 397.87 \text{ kN} \approx 398 \text{ kN}$$

The shear capacity of each bolt on channel of material Grade 43 is 12MPa.

$$n = \frac{398}{12} = 15.3 \text{ Therefore, 16 bolts are needed to resist the total shear force.}$$

And steel plates of Grade 50 are applied to connect the channels in upper and lower channels with bolts. The yield strength of steel plates of Grade 50 is 355MPa. The dimensions of the steel plates are 10mm thick, 100mm wide and 360mm long as shown in Fig. 3.1.3-9.

$$355 \times 10 \times (100 - 13 \times 2) \times 2 = 525 \text{ kN}$$

Assuming that flexural moment at the level of horizontal connection in W2 and W3 which is equal to ultimate lateral load in W1 can be resist by the connected channels. Distance from wall edge to center line of two channels at each end of the walls is 101mm including 20mm concrete cover and half of spacing the channels, 81mm. Height of wall panel is 2300mm and top beam is 480mm height. Location of the horizontal connection is at center line of wall panel as shown in Fig.3.1.3-2.

The axial compression or tension load in the channel at one end of the wall is:

$$N = \frac{429 \times (2540 - 1150)}{(1800 - 101 \times 2)} = 374 \text{ kN}$$

Since $374 \text{ kN} < 398 \text{ kN} < 525 \text{ kN}$, so the flexural connectors can withstand the compression or tension forces in channels.

b. Design shear connectors of specimen W2 and W3

Shear connectors consist of steel plates and diagonal reinforcing bars as shown in Fig.3.1.3-9 and Fig.3.1.3-10. The diagonal reinforcing bars with hook at one end is 25 deg inclined to horizon and the other end is welded on the steel plates.

It is assumed that the diagonal reinforcing bars in compression and those in tension resist the horizontal shear force simultaneously.

$$16 \times 78.5 \times 460 \cos 25^\circ = 524 \text{ kN} > 430 \text{ kN}$$

Assuming leg length of weld is 5mm, then weld length is

$$\frac{78.5 \times 460}{2 \times 0.7 \times 255 \times 5} = 20.2 \text{ mm}$$

$$\text{weld length } 20.2 + 5 + 5 \approx 30 \text{ mm}$$

According to BS8110 Part 1:1997 for concrete with cube strength 40, anchorage length of diagonal bars is $44 d_b$. In this research, the length of straight part is 500mm with a end hook.

$$f_b = F_s / \pi \phi_e l = 460 \times 78.5 / (3.14 \times 10 \times 500) = 2.3 \text{ MPa}$$

$$f_{bu} = \beta \sqrt{f_{cu}} = 0.4 \sqrt{40} = 2.5 \text{ MPa} > f_b = 2.3 \text{ MPa}$$

Where f_b is the bond stress;

F_s is the force in the bar or group of bars;

l is the anchorage length;

ϕ_e is the effective bar size which, for a single bar is equal to the bar size and, for a group of bars in contact is equal to the diameter of a bar of equal total area;

f_{bu} is the design ultimate anchorage bond stress;

β is a coefficient dependent on the bar type.

Three Grade 8.8 bolts with diameter equal to 20mm are used to connect the steel plated embedded in upper and lower wall panels.

Assuming the vertical bars in the shear connectors yield, so the vertical force at center of the wall is:

$$4 \times 78.5 \times 460 = 144kN$$

For specimen W2,

$$\sqrt{\left(\frac{144}{6}\right)^2 + \left(\frac{430}{6}\right)^2} = 75.6kN$$

For specimen W3,

$$\sqrt{\left(\frac{144}{8}\right)^2 + \left(\frac{430}{8}\right)^2} = 57kN$$

According to BS5950: Part 1:1985, the capacity of the bolt with diameter equal to 20mm on steel plate of Grade 50 and thickness equal to 10mm is 110kN.

Since $75.6kN < 110kN$, and $57kN < 110kN$ so the bolts in shear connection work.

c. Design of specimen W4

Universal beam is embedded in W4 as main vertical flexural reinforcement. The select section is UB152×89×16 as shown in Fig.3.1.3-4. The only difference between specimen W1 and W4 is different type of steel section. All other variables keep constant, including vertical and horizontal reinforcing bars, diagonal bars and strength of concrete. In design of this specimen, it is assumed that the I-beam is divided into two parts as channels in W1. The area of cross-section of I-Beam is 2050mm with yield strength 275MPa.

Similar to design of W1, the results is that the depth of neutral axis is 185mm and lateral load acting on top beam is 486kN.

d. Design of specimen W5 and W6

Specimen W5 and W6 are precast counterparts of monolithic wall W4 as shown in Fig 3.1.3-5 and Fig.3.1.3-6. W5 and W6 have the identical horizontal reinforcement with W4. W5 and W6 also have same amount the vertical reinforcing

bars and steel section to W4, whereas, the vertical reinforcing bars and steel section are cut into two. The upper and lower wall panels of W5 and W6 are assembled by connecting the horizontal connection and fill gap with concrete which has same strength as wall panel.

Similar to force transfer mechanism in the horizontal connection of W2 and W3, the I-Beams were connected to provide flexural moment resistance in the horizontal connection. The axial compression and tension in the I-Beam is

$$N = \frac{486 \times (2540 - 1150)}{1800 - 101} = 397 \text{ kN}$$

The steel plates used to connect the I-Beams embedded in upper and lower wall panels are shown in Fig 3.1.3-11 is 110 wide and 10mm thick.

$$(110 - 2 \times 13) \times 10 \times 355 \times 2 = 596 \text{ kN} > 397 \text{ kN}.$$

The detail design of flexural connectors is shown in Fig 3.1.3-11. The strength of the cross section of I-Beam is

$$275 \times (2050 - 13 \times 7.7 \times 4) = 454 \text{ kN}$$

The number of bolts:

$$n = \frac{454}{12} = 38, \text{ So 40 bolts are used in flexural connection to resist the axial}$$

force.

Since $454 \text{ kN} > 397 \text{ kN}$, therefore the flexural connectors can work.

The shear connectors used in W5 and W6 is same to those in W2 and W3, respectively. For diagonal reinforcement of the shear connector, $524 \text{ kN} > 486 \text{ kN}$, so the diagonal is strong enough to resist the horizontal force in the horizontal connection.

For specimen W2,

$$\sqrt{\left(\frac{144}{6}\right)^2 + \left(\frac{486}{6}\right)^2} = 84.5 \text{ kN}$$

For specimen W3,

$$\sqrt{\left(\frac{144}{8}\right)^2 + \left(\frac{486}{8}\right)^2} = 83 \text{ kN}$$

Since $84.5\text{kN} < 110\text{kN}$, and $83\text{kN} < 110\text{kN}$ so the bolts in shear connection work.

Fig. 3.1.3-1 Specimen W1 and Its Overall Dimensions

Fig. 3.1.3-2 Specimen W2 and Its Overall Dimensions

Fig. 3.1.3-3 Specimen W3 and Its Overall Dimensions

Fig. 3.1.3-4 Specimen W4 and Its Overall Dimensions

Fig. 3.1.3-5 Specimen W5 and Its Overall Dimensions

Fig. 3.1.3-6 Specimen W6 and Its Overall Dimensions

Fig. 3.1.3-7 Side View of All Specimens and Their Dimensions

Fig. 3.1.3-9 Details of Connection (Cont.)

Fig. 3.1.3-10 Details of Connection (Cont.)

Fig. 3.1.3-11 Details of Connection

3.2 Testing Program

3.2.1 Test Set-up

The wall specimens were tested in steel test rig was specially designed and built for the purpose. A schematic arrangement of the test set-up is shown in Fig.3.2.1-1. The general view of the test rig is given in Fig.3.2.1-2.

The specimen was fixed on the lab strong floor by fourteen high strength tension bolts with diameter equal to 50mm to prevent it from lifting. The reaction footings were placed at two ends of the base beam of the specimen to avoid horizontal sliding during loading course. Therefore, a fixed ends condition for the base beam will be simulated. There was also a special horizontal beam with a reversed “U” which was used to restrain the out-of-plane movement of top beam. The test frame is design to apply 805kN horizontal load on the test walls with a 222mm maximum horizontal travel. The jack was operated by electric hydraulic pumps. The load was applied at ends of the top beams which were stiff enough to distribute the load to the whole walls. The specimens were push to left at right end of the top beams and pull to right through loading bars at the left end of the top beams.

3.2.2 Instrumentation

The tests were monitored by load vs. displacement curve drawn in x-y coordinate, which were taken from load cell readings and horizontal displacement recorded by Linear Variable Differential Transducers (LVDT) at top beam. All these data were also recorded by data logger. LVDTs measuring horizontal displacement of top beams were fixed on a steel frame which was seated on the base beam of the specimens and the rest of LVDTs were mounted on the specimens which made LVDTs and specimens to be a unit referring to lab ground, that is, the relative movement of specimens to lab ground did not affect the horizontal displacement recorded by the LVDTs measuring horizontal displacement.

Fig. 3.2.1-1 Testing Schematic Arrangement

Fig. 3.2.1-2 General View of Set Up of Specimen W1 Fixed To The Lab Floor

Electrical resistance strain gauges were mounted on steel bars and channels or I-Beams to record the strains experience by them during the course of testing. The electrical resistance strain gauges on the steel were 5mm gauge length, $119.6 \pm 0.4 \Omega$ resistance, gage factor 2.10, adoptable thermal expansion $11.7 \text{ PPM}/^\circ\text{C}$. These strain gauges were glued to steel bars and sections with an ultra-fast adhesive and cover with silicon acting as water proof, finally a coat with strong glue was used for insulation and moisture sealing when silicon stiffened. Some of the strain gauges wires inadvertently broken and some of the gauges were damaged during the construction process due to the casting operation or vibrating procedures.

Fig. 3.2.2-1 to Fig. 3.2.2-3 show the location of the strain gauges on steel bars of specimens. Fig. 3.2.2-4 to Fig. 3.2.2-6 show the location of the strain gauges on steel sections of specimens. In these figures, a “-” is used to indicate position of the strain gauges. The difference of specimens W1 and W4 was in the location of

strain gauges on steel sections. The difference of specimens W2 and W3 is position of strain gauges on steel bars similar to specimens W5 and W6. The difference between monolithic walls W1 and W4 and their precast counterparts was the different position of strain gauges on steel bars and sections. The individual strain gauge was also tagged for identification. Strain gauges were separated into S-side and N-side. The strain gauges mounted on vertical bar were also divided into High, Middle and Low, for example, SVL1 means the strain is on S-side and low position of vertical bar 1. Strain gauges on horizontal reinforcement were noted by 1,2,3 on S-side, and 1,2,3,4,5 on N-side, for example, SH12-1 means that the strain gauge on S-side, Horizontal bar 12, number 1 counting from left. Strain gauges on diagonal bars were numerated from left to right. Strain gauges on steel section were numerated from base to top of the wall.

The positions of LVDTs are indicated in Fig. 3.2.2-7 and Fig.3.2.2-8.

The total number of LVDTs and strain gauges used in each specimen are listed in Table 3.2.2-1.

Fig. 3.2.2-1 Layout of Strain Gauges on Steel Bars Of W1 and W4

Fig. 3.2.2-2 Layout of Strain Guages on Steel Bars of W2 and W5

Fig. 3.2.2-3 Layout of Strain Gauges on Steel Bars of W3 and W6

Fig. 3.2.2-4 Layout of Strain Guages on Steel Sections of W1

Fig. 3.2.2-5 Layout of Strain Gauges on Steel Sections of W2, W3, W5 And W6

Fig. 3.2.2-6 Layout of Strain Gauges on Steel Sections of W4

Fig. 3.2.2-7 Layout of LVDTs on W1 and W4

Fig. 3.2.2-8 Layout of LVDTs on W2, W3, W5 and W6

Table 3.2.2-1 Number of LVDTs and Strain Gauges

Specimen	LVDTs	Strain Gauges	
		Steel Bars	Steel Sections
W1	12	80	28
W2	22	103	24
W3	22	103	24
W4	12	80	22
W5	22	103	24
W6	22	103	24

In order to separate the shear and flexural components which contributed to horizontal displacement of top beam of specimens, as shown in Fig.3.2.2-7 and Fig.3.2.2-8, LVDTs are aligned with diagonals of specimens to quantify the shear deformation, and LVDTs are aligned with verticals on both sides of specimens to evaluate the flexural components.

Fig. 3.2.2-9 Calculation of Shear Deformation from Rectangle

From Fig.3.2.2-9, assuming the original configuration is a rectangle, so

$$\gamma = \gamma_1 + \gamma_2 \quad (3-1)$$

$$\tan \gamma_1 = \frac{\Delta_1 / 2 \sin \alpha - \Delta_2 / 2 \sin \alpha}{L + \Delta_1 / 2 \cos \alpha + \Delta_2 / 2 \cos \alpha}, \quad (3-2)$$

$$\tan \gamma_2 = \frac{\Delta_1 / 2 \cos \alpha - \Delta_2 / 2 \cos \alpha}{H + \Delta_1 / 2 \sin \alpha + \Delta_2 / 2 \sin \alpha} \quad (3-3)$$

Where α is defined as $\tan \alpha = \frac{H}{L}$, γ is the shear deformation, Δ_1 and Δ_2 are the

deformations of diagonals in different directions and positive in extension,

respectively. L and H are the side length and height of the rectangle.

Since $\Delta_1 \sin \alpha - \Delta_2 \sin \alpha \ll L$, $\Delta_1 \cos \alpha - \Delta_2 \cos \alpha \ll L$, so

Due to the small shear deformation is considered, therefore $\tan \gamma \approx \gamma$.

$$\gamma = \frac{(\Delta_1 - \Delta_2)(\sqrt{H^2 + L^2})}{2(HL)} \quad (3-4)$$

Therefore, the shear components of total horizontal displacement of top beam is

$$u_s = \gamma H_w \quad (3-5)$$

Where u_s is component of total horizontal displacement of top beam due to shear deformation and H_w is the height of wall measured from base of the wall to the horizontal axis of the top beam.

The scheme of layout of the LVDTs aligned with verticals on both sides of the specimens is shown in Fig.3.2.2-7 and Fig.3.2.2-8, which is used to catch to flexural deformation of the walls. The deformation of vertical segments is represented by a serial of $\Delta v_{i,j}$, $\Delta v_{l,j}$ and $\Delta v_{r,j}$ indicate vertical deformations of these segments on left and right ends of the specimens, respectively. When the top beam displacement of the wall, Δu , is negative when wall is pushed to the left. In order to attain the curvature of different segments of the wall, the rotation of these segments should be available. For example, the rotation of line 2 relative to line 1 is calculated by

$$\theta_2 = \frac{\Delta v_{l,2} - \Delta v_{r,2}}{l_w} \quad (3-6)$$

where the l_w is the distance between both rows of the LVDTs.

The curvature in the interval between line1 and line2 on the center can be calculated as

$$\phi_2 = \frac{\theta_2}{\Delta_2} \quad (3-7)$$

Where θ_2 is the relative rotation of line1 to line 2, ϕ_2 is the curvature segment of wall between line1 and line2. The average gauge length Δ_2 can be can attained by

$$\Delta_2 = \frac{(v_{l,2} + \Delta v_{l,2}) + (v_{r,2} + \Delta v_{r,2})}{2} \quad (3-8)$$

The formula of rotation and curvature of the different segments of wall can be achieved by changing the index 2 by 1, 3, 4 and so forth.

The rotation angle determined by equation 1 is a relative rotation angle between two adjacent lines. Therefore, in order to obtain the flexural horizontal displacement of the top beam, a fixed horizontal line needs to be assumed to be a reference. Because all these LVDTs were mounted vertically along the ends of the specimens and no addition fixing frame was used to fix them. Yanez (1993) discussed the determination of the fixed line. Yanez suggested that the flexural deformations $\Delta v_{l,1}$ and $\Delta v_{r,1}$ be not correctly evaluated if these deformations was assumed only occurred above the base line of the wall. So the curvature of the first interval will be fault. Through checking the gauges adherent below the base line, horizontal line 50mm below the base line was taken as the fixed line. Therefore, in present research, the line 50mm below the base line is used as the reference for the first interval. This accounted for strain penetration into the base beam. The flexural deformation components of horizontal displacement of top beam can be approximately evaluated with $\theta_1, \theta_2, \theta_3$ and θ_4 .

$$\Delta = \Delta_1 + \Delta_2 + \Delta_3 + \Delta_4 = \theta_1 H_1 + \theta_2 H_2 + \theta_3 H_3 + \theta_4 H_4 \quad (3-9)$$

Where H_1, H_2, H_3 and H_4 are the distance from center of these intervals to the horizontal axis of the top beam.

Fig. 3.2.2-10 Flexural Deformation from Verticals at Both Ends of Specimens

Fig. 3.2.2- 11 Calculation of Flexural Deformations

3.2.3 Load history

The wall specimens were load with slow cycles of lateral load in two stages. The first stage involved three reversed load-controlled cycles in the elastic range at 0.25, 0.5 and 0.75 times the nominal maximum load (ideal strength), respectively. The ideal or nominal strength of a section of a member, S_i , is derived from the dimensions, reinforcement content, and details of the section designed, and code-specified nominal material strength properties, suggested by Paulay (1992). The second stages involved reversed displacement-controlled cycles of loading at increasing level of displacement ductility factor (DF) until the load carrying capacity of the wall was exhausted. As the effect of loading history was not selected as a test variable, the typical displacement-control load was applied two cycles at each displacement level. The displacement ductility factor (DF) was defined as the ratio of the lateral displacement at any step to the displacement at yield. The yield displacement Δ_y was estimated as 1.33 times the average of the positive and negative displacements corresponding to loads of 0.75 of the ideal strength of the wall in both directions, respectively, as shown in Fig. 3.2.3-1. The tests were ended when the strength decreased to less than 80% of the recorded maximum strength.

These criteria for the displacement controlled cycles are based on a reduced stiffness equivalent elasto-plastic yield and on a significant load capacity after peak load suggested by Park (1989).

The load history applied for all the specimens is given in Fig. 3.2.3-2 to Fig. 3.2.3-7. In these figures, scan numbers are used as x-axis and displacement is used for y-axis. In course of testing, different load stages were shown in photographs including load-controlled and displacement-controlled stages.

Fig. 3.2.3-1 Definition of yield displacement (Park, 1989)

Fig. 3.2.3-2 Load history of specimen W1

Fig. 3.2.3-3 Load history of specimen W2

Fig. 3.2.3-4 Load history of specimen W3

Fig. 3.2.3-5 Load history of specimen W4

Fig. 3.2.3-6 Load history of specimen W5

Fig. 3.2.3-7 Load history of specimen W6

3.3 Summary

In this chapter, total six specimens, two monolithic and four precast walls, are designed. In order to grasp critical features of specimens during loading, a number of LVDTs and strain gauges are used in this research and they are laid out in appropriate locations. Referring to previous studies on shear walls by other researchers, loading course is given.

CHAPTER 4

EXPERIMENTAL RESULTS

All the specimens were tested to failure under reversed cyclic load as previously described. The focus of herein is to summarize the qualitative and quantitative experimental results. Photographs is taken to record the qualitative results through the course of testing, such as, cracking patterns, crushing locations, bar breaking, fracture of steel sections and failure modes. Readings of LVDTs, strain gauges and crack widths were also used in this research to catch the quantitative behavior of walls.

Load-displacement response and cracking patterns of the walls were the mainly features of the results. Other characteristics including stiffness, curvature, flexural shear and sliding deformation, vertical elongation and failure modes of the specimens are presented in this chapter.

4.1 Loading Procedure

4.1.1 Load and Displacement Histories

Horizontal displacement versus scan number diagrams as shown in Fig.4.1.1-1 to Fig.4.1.1-6 show the histories of the horizontal displacement of the top beam of specimens in the courses of testing. Similarly, Fig.4.1.1-7 to Fig.4.1.1-12 show the histories of horizontal load affecting on the horizontal axis of the top beam. As loading diagram schemed before, the loading course was divided into force-control and displacement-control. In elastic stage, load was applied by force-control in sequence, $0.25 P_i$, $0.5 P_i$, $0.75 P_i$ and P_i , and one cycle was for each load level in this stage. In the plastic stage, two displacement-control cycles were applied for each displacement level. Therefore, the horizontal force in the second stage taken from load cell was results from horizontal displacement at different ductility levels. Figures of histories of lateral load showed that the maximum strength kept almost constant when specimen entered the plastic stage and that the maximum

strength in the second cycle at each displacement level was a little lower than that of first cycle. When displacement reached a level, that is, after a plastic plateau the strength of the specimen started to decrease.

4.2 Experimental Results

All these specimens were designed, ntested to failure following the above-mentioned loading histories in the heavy structures laboratory at Nanyang Technological University, Singapore. Reading of strain gauges placed on vertical, horizontal and diagonal reinforcement and steel sections were collected. And records of LVDT in the loading course were also applied in this part to analyze behavior of these specimens.

Fig. 4.1.1-1 Horizontal Displacement History of Specimen W1

Fig. 4.1.1-2 Horizontal Displacement History of Specimen W2

Fig. 4.1.1-3 Horizontal Displacement History of Specimen W3

Fig. 4.1.1-4 Horizontal Displacement History of Specimen W4

Fig. 4.1.1-5 Horizontal Displacement History of Specimen W5

Fig. 4.1.1-6 Horizontal Displacement History of Specimen W6

Fig. 4.1.1-7 Lateral Load History of Specimen W1

Fig. 4.1.1-8 Lateral Load History of Specimen W2

Fig. 4.1.1-9 Lateral Load History of Specimen W3

Fig. 4.1.1-10 Lateral Load History of Specimen W4

Fig. 4.1.1-11 Lateral Load History of Specimen W5

Fig. 4.1.1-12 Lateral Load History of Specimen W6

4.2.1 Load-Displacement Response

Load-displacement responses of all specimens are shown in Fig.4.2.1-1 to Fig.4.2.1-6. The typical characteristics of the reinforced concrete members subjected to reversed cyclic load, such as unloading and reloading stiffness reduction as the cyclic displacement level increases, and pinching of hysteresis loops, can be clearly seen from these figures. Low height-to-width shear walls generally produce hysteresis curves that are highly pinched and exhibit significantly less energy dissipation than would similar walls with a larger height-to-width ratio. More energy is dissipated through the structure in the post-peak range as the concrete begin to soften and the reinforcement approaches yielding.

The unloading curves of the second excursion at each displacement level generally followed the same unloading path as the first. The reloading branches of the second excursion of displacement followed a similar loading path as the first, but a lower stiffness, resulting in lower peak strengths. The load-displacement

curves indicated that the first excursion of a new displacement level followed the loading path of the second excursion of the previous displacement level. This suggests that additional cycles at a specific displacement level would produce negligible damage in comparison to that experienced by the first unloading-reloading cycle. The responses demonstrated this trend until the peak load. During the post-peak ranges, the amount of damage experienced during subsequent cycles was significant and similar to that sustained during the second excursion of the previous displacement. The increased accumulation of damage was a function of the concrete's inability to contribute significantly the lateral resistance.

The hysteresis loops show that all precast specimens except W6 exhibited good performance in comparison with their monolithic prototypes including maximum strength and ultimate displacement which are also indicated by Fig.4.2.1-1 to Fig.4.2.1-6. However, the ductility of the precast walls is smaller than that of their monolithic prototypes. Some specimens only resist one cycle at the maximum displacement level whereas others resist two cycles at the maximum displacement level when the strength dropped below 75% of the maximum experimental strength.

The horizontal connections in precast walls almost made no difference in yielding strength and ultimate strength in comparison with their monolithic prototypes, which can be also seen from Fig.4.2.1-1 and Fig.4.2.1-2. This verified the idea of connection design, using the embedded steel sections to carry the bending moment.

The yield displacement of the precast specimens occurred at larger displacement compared to that in monolithic prototypes which was one source of the lower ductility of the precast walls. One reason for postponed yield point is due to the tolerance between the bolt hole and bolt in the flexural and shear connection which allowed a very small rocking of the upper precast wall panel and a similar sliding along the horizontal connection. The other was due to the discontinuity of the vertical reinforcement at the horizontal interface between the precast wall segments which decreased the stiffness of horizontal connection. From

experimental result, the precast walls exhibited lower ultimate displacement. As a result, the ductility of precast was lower than that of those monolithic walls.

Pinching effect in precast walls was more serious than that in monolithic ones, which indicated lower capacity of energy dissipation of the precast walls. The extent of pinching is a hint of crack closing and consequent loss of stiffness during reloading in the opposite direction. One cause is the closing of the opened crack under previous loading. For precast wall, due to existence of horizontal connection, the sliding of the upper wall panel along the horizontal connection during loading is also a reason and the closing of the horizontal connection is another root.

Fig. 4.2.1-1 Hysteresis Loops of Specimen W1

Fig. 4.2.1-2 Hysteresis Loops of Specimen W2

Fig. 4.2.1-3 Hysteresis Loops of Specimen W3

Fig. 4.2.1- 6 Hysteresis Loops of Specimen W6

4.2.2 Backbone Envelop of Load-Displacement Response

Fig.4.2.2-1 to Fig.4.2.2-2 show the backbone envelopes of the load-displacement response of these walls. The envelope response confirmed the difference between strength and stiffness between the monolithic walls and their precast counterparts.

The comparison of monolithic wall with embedded channels and its precast counterparts is shown in Fig. 4.2.2-1. It indicates that the lateral load-resistant capacity of precast counterparts is almost equal to that of their monolithic prototypes. However, the initial stiffness of these walls is decreased W1, W3, W2, in sequence. This means that the monolithic wall has the highest initial stiffness and the wall with two shear connectors has a little higher initial stiffness in comparison with that of wall with one shear connector. However, the difference in strength and stiffness between the monolithic wall and its precast counterparts is not remarkable. This hints that the horizontal connection can successfully transfer forces from upper wall panel to the lower one in precast walls without losing much strength and initial

stiffness. Nevertheless, the precast walls showed total different failure mode and different failure locations, which are discussed in subsequently sections.

Fig.4.2.2-2 shows the backbone envelopes of load-displacement response of the walls with embedded I-Beams. The characteristics of these walls exhibit the similar behaviors as those of walls with embedded channels, except that specimen W6 experienced connection failure. During load cycles spalling of concrete cover occurred in walls with embedded I-Beam along inner boundaries of I-Beams, which did not appear in walls with channels. The monolithic wall showed flexural ductile behavior whereas precast walls exhibited sudden failure. The causes are discussed in following sections.

Fig. 4.2.2-1 Backbone Envelopes of Load-Displacement Response of Specimens

Fig. 4.2.2-2 Backbone Envelopes of Load-Displacement Response of Specimens

4.2.3 Stiffness

A typical nonlinear relationship between induced forces or loads and displacements, describing the response of a reinforced concrete component subjected to monotonically increasing displacements, is shown in Fig.4.2.3-1. And for purpose of routine design, the observed response curve is normally replaced by two bilinear approximations. The slope of the idealized linear elastic response, $K = S_y / \Delta_y$, is used to quantify stiffness. This should be based on the effective secant stiffness to the real load-displacement curve at a load of about $0.75 S_y$. The definition of stiffness of reinforcement concrete member under cyclic load is the initial curved load-displacement characteristic will modify to close to the linear relationship of the idealized response, Paulay (1992).

As recommended by Park (1988), in order to estimate the ductility level which specimens fail, the yield displacement was defines as that corresponding to the point of intersection of the horizontal line passing through the higher points of the envelope curve and the inclined line passing through the beginning of the axes

and point of envelop curve corresponding 75% of the maximum strength. It was considered that failure of the specimen occurs when its strength is decreased to 75% of its maximum strength. The final value for displacement ductility is the ratio of the displacement for a strength decrease up to 75% of the maximum strength, divided by the yield displacement.

Fig. 4.2.3- 1 Typical Relationship for A Reinforced Concrete Element
(Paulay and Priestley 1992)

The initial stiffness of the precast walls is lower than that of their prototypes. This is due to the postponed yield points in precast walls. The major reason is the built-in tolerance between bolt and bolt holes in the horizontal connection. The stiffness is mainly depends on the number of the shear connectors as compared in Fig.4.2.2-2. The initial stiffness is calculated by

$$K_i = S_y / \Delta_y$$

Where K_i is the initial stiffness of the specimens, S_y is yield strength of specimen, approximately equal to S_i , in Table.3.2.5-1, S_y is taken from experimental results and not equal to S_i , Δ_y is the yield displacement obtain from the experimental results.

Fig.4.2.3-2 shows the relationship of lateral stiffness versus drift ratio, which calculated from backbone envelope of load-displacement. It shows that in the elastic range the stiffness is highest and decrease with increasing drift ratio. It also shows that the fastest decrease of lateral stiffness is in the elastic range. With increasing drift the slope of lateral stiffness declines which means that the decrease in stiffness becomes slow. It shows that at same drift ratio the later stiffness of monolithic walls W1 and W4 exhibit higher stiffness in comparison with their own precast counterparts.

Fig. 4.2.3-2 The relationship of Lateral Stiffness versus Drift Ratio

4.2.4 Cracking Patterns

The cracking patterns are important data to internal load paths and failure modes. General features of the cracking patterns of all specimens are presented in this section. The particular features of the each specimen are studied in later sections.

Cracking patterns of the specimens are shown in Fig.4.2.4-1 to Fig.4.2.4-6. The cracking patterns presented herein at DF equal to 1 and cycle number equal to 4 in elastic stage, and DF equal to 2 and cycle number equal to 5 and minimum ductility factor in plastic stage. The cycle 5 is the first cycle in plastic stage and ductility increases from 1 to 2, which means the transition from elastic stage to plastic stage. The increment of number of additional cracks in this cycle is the largest in any single loading cycle, which corresponds to prominent jump of readings of the strain gauges on reinforcement shown in later sections, especially in lower parts of the specimens.

Specimen W1 exhibited a flexural behavior during the test which finally showed a concrete crushing failure at right lower corner with breaking weld of the lowest horizontal reinforcement and buckling of the channels. The horizontal cracks occurred in first cycle at the location of the horizontal reinforcement and propagated from boundary downward to center of the wall. Near the boundaries the cracks about 200mm length is nearly horizontal, while further to center they were inclined to the horizontal. The horizontal cracks are almost evenly distributed, along at the location of the horizontal reinforcement. The diagonal cracks occurred in the upper part of the wall, direction of these diagonal cracks were along a diagonal line from top corner of the wall to the opposite bottom corner and they joined the flexural cracks at the boundaries. With increasing lateral load, the density of the horizontal and diagonal crack in central part of the wall became higher. And width of diagonal concrete bands formed between two adjacent diagonal cracks became smaller and these bands finally showed an even distribution with a width about 200mm as shown in Fig.4.2.4-1. When ductility reached 5, vertical cracks also appeared in lower corners, which indicated spalling of concrete cover at these positions. Meanwhile, a lower inclined crack along the diagonal reinforcement opened considerably more than others and after several cycles, concrete at this place was cracked with dense spacing, which showed as critical region, especially at ends of the diagonal rebars, but in subsequent cycles, further deterioration did not occur. Following spalling at the lower corners of the specimen, large permanent deformation occurred in the vertical bars, which finally caused fracture of the

outermost vertical bars at both ends of the wall. This can be considered as the failure stage, although no abrupt drop in strength was observed. A wide horizontal crack as shown in cycle 15 occurred along the intersection between wall panel and base beam which not appeared in its precast counterparts. A wide horizontal crack as shown in cycle 15 occurred along the intersection between wall panel and base beam which not appeared in its precast counterparts. Subsequent to this stage, sliding at the base was observed. Later with a sound of breaking weld of the lowest horizontal reinforcement, buckling of channels occurred accompanied by concrete crushing at the lower corner. Length of the horizontal flexural crack at two ends of the wall almost linearly decreased to zero from bottom to top of the wall with decreasing bending moment in horizontal sections. In this specimen, in contrast to the requirement on diagonal reinforcement for avoiding sliding shear failure which requires diagonal reinforcement should cross all sections within a distance $0.5 l_w$ or $0.5 H_w$, which is less, above the base section (Paulay and Priestley, 1992), the height of the sections with diagonal bars was only $0.31 l_w$, which also successfully prevented the sliding shear failure.

Cracking patterns of specimen W2 and W3 which are precast counterparts of specimen W1 are shown in Fig.4.2.4-2 and Fig.4.2.4-3. Before lateral load reached half of capacity of the wall, the diagonal cracks mainly occurred in lower panel of the wall and the inclination of these cracks was lower than those in specimen W1. After lateral load reached $0.75 P_i$, there was a horizontal crack passing through the horizontal connection. When ductility reaches 1, density of diagonal cracks is higher than that in specimen W1. Almost all main cracks were produced when ductility increased from 1 to 2, which is a basis of internal load paths of the wall. The distribution of horizontal cracks is different from those in specimen W1, especially, the density of them is high near the connections which means relative large deformation occurred in this zone. From the cracking patterns of specimen W2, the diagonal cracks in upper and lower panels look discontinuous, especially cracks in upper panels. The horizontal crack at the intersection of the wall panel and base beams which existed in specimen W1 did not appear in

specimen W2. The inclination of diagonal in upper and lower panels is lower compared with that in specimen W1. In late cycles two considerable wide shear cracks in opposite directions appeared in along the lower diagonal reinforcement, these cracks shaped as “V” with an obtuse angle. There are a lot of cracks with high density, which mainly distributed in the lower panel along the inner channel embedded in the wall. These dense cracks made concrete in this zone with dense spacing, and finally caused spalling of concrete cover in last several cycles. The spalling of concrete is serious and finally the specimen W2 failed in shear with breaking of horizontal reinforcement passing through the spalling zone and buckling of channel at this location. Specimen W3 failed with fracture of channel near the connection with spalling of concrete and breaking of horizontal reinforcement passing through this region.

Specimen W4 is a wall with I-Beam embedded at two ends acting as main flexural reinforcement. Before ductility reached 1, the cracking patterns of W4 including distribution of horizontal cracks at ends and diagonal cracks in central part of the wall was similar to W1, except that some vertical cracks appeared in lower part of specimen W4 along the inner boundaries of I-Beam. With increasing lateral load, more diagonal cracks occurred in direction from top corner of wall to opposite lower corner. The almost vertical cracks along the inner boundaries of I-Beam extended to higher place. Later these cracks caused spalling of concrete cover in this location as shown in cycle 17 in Fig.4.2.4-4. Diagonal cracks with lower inclination occurred in upper part of the wall. These cracks almost passed through the center of top beam, which did not happen in specimen W1. Vertical cracks occurred at lower corner of the wall are similar to that in specimen W1, which means spalling of the concrete cover at this position. In late several cycles, spalling of concrete along the inner boundaries of I-Beam is serious from bottom to top of the wall. When the lateral load reversed, a frictional sound can be heard. Finally, the wall failed in flexural behavior with crushing concrete at left lower corner. Compared with crack patterns of specimen W1, the distribution of diagonal cracks of specimen W4 is more evenly which means the horizontal reinforcement in higher part of specimen W4 experienced higher strain than those in specimen W1.

Specimen W5 and W6 are precast counterparts of specimen W4. Crack patterns of specimen W5 are shown in Fig.4.2.4-5. In early loading cycles before the lateral load reached P_i , the crack patterns is similar to that of specimen W4 with diagonal cracks passing through horizontal connection from top of the wall to bottom. Inclination of diagonal cracks in upper panel is lower than that in specimen W4. Density of diagonal cracks in lower panel is apparently higher than that in upper one. Width of the diagonal crack bands in lower wall panel is about 100mm or less whereas it is about 200mm in upper panel. When ductility is greater than 3, almost no additional cracks were produced. With increasing ductility, two main diagonal cracks in each direction appear along the diagonal reinforcement similar to those in specimen W2 and W3 and spalling of concrete cover occurred at the upper ends of diagonal bars. In late cycles, spalling of concrete cover along the inner boundaries of I-Beam which happened in specimen W4 also occurred, however, the vertical range is different. In specimen W4 the spalling of concrete occurred almost in all height of the wall, whereas in specimen W5, spalling is in range of the lower panel. Under reversed cyclic load, spalling continued to form two triangular zones, which are more serious than that occurred in specimen W2 and W3. And concrete crushing occurred at left lower corner of the wall. The specimen failed in shear with breaking of the horizontal reinforcement passing through the serious spalling zone, which is different from the fail mode of its prototypes specimen W4 which failed in flexure.

Fig. 4.2.4-1 Cracking Patterns of Specimen W1

DF=1 Cycle 4

DF=2 Cycle 5

DF=3.5 Cycle 9

Fig. 4.2.4-2 Cracking Patterns of Specimen W2

DF=1 Cycle 4

DF=2 Cycle 5

DF=4 Cycle 9

Fig. 4.2.4- 3 Cracking Patterns of Specimen W3

Fig. 4.2.4-4 Cracking Patterns of Specimen W4

Fig. 4.2.4-5 Cracking Patterns of Specimen W5

Fig. 4.2.4-6 Cracking Patterns of Specimen W6

A summary of the characteristic of the experimental results is shown in Table 4.2.4-1.

4.2.5 Flexural and Shear Deformations

The total displacement of the walls was monitored by LVDT mounted on steel frame, which is composed of flexural and shear deformations and sliding of the walls relative to its base beam. To study the contributions of flexure and shear components, the total horizontal displacement is decomposed into shear and flexure and their contribution to total displacement is presented by ratios to total displacement separately. Flexural deformations is evaluated by the sum of top horizontal displacements caused by rotations of the intervals and shear deformation can be obtained through measuring the changes occurred in the two diagonal LVDTs as discussed in previous sections. All assumptions made in calculation have been explained in section 3.2.2.

Table 4.2.4-1 Characteristic of Experimental Results

	W1		W2		W3		W4		W5		W6	
	(-)	(+)	(-)	(+)	(-)	(+)	(-)	(+)	(-)	(+)	(-)	(+)
S_y (kN)	522.8		522.8		522.8		608.9		608.9		608.9	
Δ_y (mm)	8.57	10.91	12.59	14.11	10.99	11.47	6.61	9.31	12.24	10.07	-	-
Δ_y -aver (mm)	9.74		13.35		11.23		7.96		11.16		-	
S_u (kN)	570.29	554.28	576.96	541.6	601	547	785.73	728.36	749.4	731.7	723.7	-
Δ_u (mm)	53.67		52.02		45.99		58.65		53.67		16.8	
Drift (%)	2.11		2.05		1.81		2.31		2.11		-	
μ	5.5		3.9		4		7.4		4.8		-	
K_i (kN/mm)	58.33		39.16		46.56		77.4		55.12		-	
γ_{sy}	-		0.99		1.02		-		0.97		-	
$\gamma_{\Delta y}$	-		1.3		1.2		-		1.35		-	
γ_{su}	-		0.99		1.02		-		0.97		-	
$\gamma_{\Delta u}$	-		0.97		0.856		-		0.92		-	
Failure Mode	Flexural		Shear		Shear		Flexural		Shear		Connection	

Where S_y is the yield strength, Δ_y is the horizontal displacement at yielding, S_u is the ultimate strength of the specimen, Δ_u is the ultimate displacement of specimen at failure, drift is defined as the ratio of Δ_u / H_w , H_w is the height of the wall measured from base line of wall to the horizontal axis of the top beam. μ is ductility factor defined as the ratio of Δ_u / Δ_y , K_i is the initial stiffness defined as the ratio of S_y / Δ_y , γ_{sy} is the ratio of yield strength of precast walls to that of their monolithic counterpart, $\gamma_{\Delta y}$ is the ratio of yield displacement of precast walls to that of their monolithic counterparts, γ_{su} is the ratio of ultimate strength of precast walls to that of their monolithic counterparts, $\gamma_{\Delta u}$ is the ratio of ultimate displacement of precast walls to that of their monolithic counterparts.

Flexural and shear deformations of each specimen under cyclic load are presented in Fig.4.2.5-1 to Fig.4.2.5-10. The total horizontal displacements of these specimens are decomposed into shear and flexural deformations which are shown in Fig.4.2.5-11 to Fig.4.2.5-15. Ratios of flexural deformation and shear deformation

to the total horizontal displacements during the loading courses are also given in Fig.4.2.5-16 to Fig.4.2.5-20, which are used to judge failure mode of each specimen.

The hysteresis loops of flexural deformations versus lateral load of monolithic walls, as shown in Fig.4.2.5-1 and Fig.4.2.5-7 are much fatter than those of shear deformations given in Fig.4.2.5-2 and Fig.4.2.5-8, which means that the energy during test is mainly dissipated through flexural deformations. The hysteresis loops of flexural deformations against lateral load of precast specimens is also fatter than their corresponding hysteresis loops of shear deformations. But these hysteresis loops of flexural deformations of precast walls are narrower in comparison with those of their monolithic counterparts, which means that less of energy is dissipated through flexural deformations in precast walls.

Compared with deformations occurred in their monolithic prototypes, in precast walls, flexural components of total deformation is less at the same displacement level in plastic stage as shown in Fig.4.2.5-11 to Fig.4.2.5-15 than that in monolithic prototypes. This can be also inferred from the different cracking patterns in monolithic prototypes and their precast counterparts as shown in Fig.4.2.5-1 to Fig.4.2.5-5. The density and width of horizontal cracks at two ends of monolithic walls are higher than that in precast walls and there is also a wide horizontal crack go across along the base beam where bond slip occurred.

In precast specimens, W2, W3 and W5, bigger portion energy dissipated through shear deformation than that in monolithic walls. The energy dissipated through shear deformation mainly came from plastic deformation of horizontal reinforcement and diagonal bars.

The hysteresis loops of shear deformation of monolithic walls, W1 and W4, exhibit less pinching as compared with the walls without diagonal reinforcement predicted by FEM in later chapter and this phenomenon is consistent with prediction by Sittipunt and Wood (1995).

The ratio of flexural component to the total deformation increases during the last cycle in specimens W1 and W4, which indicates that the increment of flexural deformation is dominating in this stage. This increment indicates that the walls tend to flexural failure, which is consistent with the observed failure modes of the two specimens. However, in precast specimens, W2, W3 and W5, ratios of shear deformation to total horizontal displacement increase while the proportion of flexural deformation decrease in plastic stage, and in last cycle, a sudden increment occurred in ratio of shear deformation which means that the specimens experienced a shear failure. This is consistent with failure modes of these precast specimens which were observed in experiment.

Fig. 4.2.5- 1 Flexural Deformation of Specimen W1

Fig. 4.2.5-2 Shear Deformation of Specimen W1

Fig. 4.2.5-3 Flexural Deformation of Specimen W2

Fig. 4.2.5-4 Shear Deformation of Specimen W2

Fig. 4.2.5-5 Flexural Deformation of Specimen W3

Fig. 4.2.5-6 Shear Deformation of Specimen W3

Fig. 4.2.5-7 Flexural Deformation of Specimen W4

Fig. 4.2.5-8 Shear Deformation of Specimen W4

Fig. 4.2.5-9 Flexural Deformation of Specimen W5

Fig. 4.2.5-10 Shear Deformation of Specimen W5

Fig. 4.2.5-11 Lateral Loads versus Deformations of Specimen W1

Fig. 4.2.5-12 Lateral Loads versus Deformations of Specimen W2

Fig. 4.2.5-13 Lateral Loads versus Deformations of Specimen W3

Fig. 4.2.5-14 Lateral Loads versus Deformations of Specimen W4

Fig. 4.2.5-15 Lateral Loads versus Deformations of Specimen W5

Fig. 4.2.5- 16 Ratios of Deformations to Total Displacement of Specimen W1

Fig. 4.2.5-17 Ratios of Deformations to Total Displacement of Specimen W2

Fig. 4.2.5-18 Ratios of Deformations to Total Displacement of Specimen W3

Fig. 4.2.5-19 Ratios of Deformations to Total Displacement of Specimen W4

Fig. 4.2.5-20 Ratios of Deformations to Total Displacement of Specimen W5

4.2.6 Vertical Elongation of the Walls

Different numbers of LVDTs were placed on both ends of these specimens to measure vertical deformation, that is, vertical elongation at these local positions. These vertical elongations can be used to determine the extent of the cracking and yielding of the flexural reinforcement. Cycles beyond yielding lead to a ratchet effect. This describes the vertical stretching of the wall caused by the irrecoverable strain that accumulated in the post-yield cycles. Records of LVDTs mounted on left side of specimens W1 are shown in Fig.4.2.6-1 and Fig.4.2.6-6. Significant elongations of the end of the wall, while in tension, are evident, due to flexural cracking and the straining of the reinforcement at the cracks. The elongation of the ends increased up to the peak load.

Fig.4.2.6-1 to Fig.4.2.6-4 exhibit the local elongations in different vertical intervals of specimen W1. The peak values of local elongations show the trend to increase from top to bottom. The relatively apparent residual deformations measured by LVDTs on left side of the wall when lateral load is zero indicate that the yielding of flexural reinforcement at end of the wall was widespread. Fig.4.2.6-6

shows a big part of the total elongation of the wall is taken up by elongation in the lowest interval, which indicates vertical elongation mainly concentrates in lowest interval. This is also indicated by the cracking patterns of specimen W1 shown in Fig.4.2.4-1 in which the density of flexural cracks, that is, horizontal cracks at two ends of the specimen, is higher in lower segment than that in upper part of the wall. This verified that the accumulated flexural cracks and plastic straining of reinforcement consists the vertical elongation.

In the reinforced concrete structures, the main capacity of energy dissipation is contributed by plastic deformation of reinforcement. The records of the LVDTs can be used to estimate the deformation of the vertical reinforcement, which was also evaluated from readings of strain gauges placed on reinforcing steels discussed in detail in subsequent sections. Therefore, it can be concluded that a key part of energy dissipation that occurred during the reversed cyclic loading test was from lower 1/4 segment of the specimen W1 where plastic hinge formed.

Vertical deformations of the different vertical intervals of the precast specimen W2 and total vertical elongation are shown in Fig.4.2.6-7 to Fig.4.2.6-13. In this specimen, as shown in Fig.4.2.6-9, the local deformation in the interval covering the horizontal connection comprises the main part of the total elongation, whereas in specimen W1 the local elongation in the lowest segment does. In specimen W2, the local elongation in the lowest interval is smaller in comparison to that in specimen W1. The cracking patterns of the specimen W2 as shown in Fig.4.2.4-2 hint these characteristics by the density of the flexural cracks at the two ends of the wall, which means that the energy dissipation concentrates in the region of the horizontal connection, whereas the energy dissipation in the lowest segment of the wall W2 was not so much as that in specimen W1. Though the total vertical elongation that occurred in specimen W2 was higher than that of specimen W1, considering the capacity of energy dissipation deduced from the configuration of the hysteresis loops, the capacity of specimen W2 exhibited similar or a little lower energy dissipation than that of specimen W1. The reason was that not all of the local vertical deformation in the interval covering the horizontal connection comes

from deformation of the steel reinforcement. As mentioned early, the built-in tolerance between the bolts and bolt-holes in the horizontal connection was a key variable of the local elongation. In specimen W2, in last two cycles the local deformation of the second lowest interval was high which corresponded to the failure location observed in experiment.

Fig.4.2.6-14 to Fig.4.2.6-20 show the characteristics of vertical elongations of the specimen W5. The deformations of specimen W5 which is a precast wall with embedded I-Beam exhibit similar characteristics of specimen W2. The local elongation in the lowest interval was smaller compared with that in the interval covering horizontal connection and also smaller than that in specimen W4 in which there is a wide horizontal crack occurred along the base line. The elongation in the interval covering horizontal connection is the main contribution to the total vertical elongation similar to that in specimen W2.

The maximum values of total vertical elongation of each specimen are listed in Table 4.2.6-1. And the ratios of maximum vertical elongations to their heights are also given in this table, which shows that the precast walls experienced higher vertical deformations than their own monolithic prototypes. This will cause higher rotation demand on other structure elements connected to the wall. However, elongation analysis showed that the main vertical elongation of W2 occurred in the connection. It indicated that the elongation could be reduced by improve the performance of the connection, for example, using high strength friction bolts to replace the normal bolts.

Fig. 4.2.6-1 Local Vertical Elongations of W1-LVDT2 (cont.)

Fig. 4.2.6- 2 Local Vertical Elongations of W1-LVDT3 (cont.)

Fig. 4.2.6-3 Local Vertical Elongations of W1-LVDT4 (cont.)

Fig. 4.2.6-4 Local Vertical Elongations of W1-LVDT5

Fig. 4.2.6-5 Total Vertical Elongations of W1

Fig. 4.2.6-6 Vertical Elongations of W1 versus Scan Number

Fig. 4.2.6-7 Local Vertical Elongations of W2 (cont.)

Fig. 4.2.6-8 Local Vertical Elongations of W2 (cont.)

Fig. 4.2.6-9 Local Vertical Elongations of W2 (cont.)

Fig. 4.2.6-10 Local Vertical Elongations of W2 (cont.)

Fig. 4.2.6-11 Local Vertical Elongations of W2 (cont.)

Fig. 4.2.6-12 Vertical Elongation of Specimen W2

Fig. 4.2.6- 13 Vertical Elongations of W2 versus Scan Number

Fig. 4.2.6-14 Vertical Elongation of Specimen W5

Fig. 4.2.6-15 Vertical Elongation of Specimen W5 (Cont.)

Fig. 4.2.6-16 Vertical Elongation of Specimen W5 (Cont.)

Fig. 4.2.6-17 Vertical Elongation of Specimen W5 (Cont.)

Fig. 4.2.6-18 Vertical Elongation of Specimen W5

Fig. 4.2.6-19 Vertical Elongations of W5

Fig. 4.3.6-20 Vertical Elongations of W5 versus Scan Number

Table 4.2.6-1 Vertical Elongation of Specimens

Specimen	ΔH (Max. Vertical Displacement mm)	$\Delta H/H$ (%)
W1	21.83	0.95
W2	57.72	2.51
W3	52.86	2.3
W4	30.36	1.32
W5	49.77	2.16

Note: H=2300mm,

ΔH =average of sum of records of LVDTs on left and right sides when lateral load is zero.

4.2.7 Curvature

Generally rotation is the most favorable inelastic structural deformations in the reinforced concrete structures. It is based on the tension of reinforcement and compression of concrete on the other side of the RC member and will not lead to a sudden failure. The maximum rotation occurs in the plastic hinge when the structure reaches post-yield stage. In this study, in order to investigate the overall behavior of the specimens, curvatures along the height of the walls are given. The horizontal axis is the distance from the base beam. The vertical axis is calculated curvature, which is plotted in the middle of the intervals determined by the gauge lengths between adjacent LVDTs.

The curvatures of each specimen are obtained from the records of LVDTs placed on both sides of the specimens. The calculation of curvature is explained in previous section. Curvatures at peak loads in first cycle at each displacement level are adopted.

As shown in Fig.4.2.7-1 and Fig.4.2.7-4, in the monolithic walls the rotation mainly concentrates in 1/4 height of the wall. This is consistent with experimental observation. However, as given in Fig.4.2.7-2, Fig.4.2.7-3 and Fig.4.2.7-5, in precast walls the rotation extends to 2/3 height of the specimens. And the rotation

occurred in the intervals covering the horizontal connections is high, but this can not be considered as the favorable deformation in plastic hinge, because this mainly comes from the built-in tolerance between bolts and their wholes and discontinuity of the vertical bars.

Fig. 4.2.7-1 Curvature of Specimen W1

Fig. 4.2.7-2 Curvature of Specimen W2

Fig. 4.2.7-3 Curvature of Specimen W3

Fig. 4.2.7-4 Curvature of Specimen W4

Fig. 4.2.7-5 Curvature of Specimen W5

4.2.8 Sliding of the Walls

Sliding of each wall during cyclic loading course was monitored by the LVDT placed above 250mm above the base line. Sliding shear deformations generally develop at the height at which vertical reinforcement yields. It is proposed by Penelis (1997) that the length of the plastic hinge is according to the relation $l_p = 0.08l_0 + 6\phi$ where l_0 is the element's length from the point of zero moment, to the point of maximum moment, and ϕ is the diameter of the largest steel bar. Fig.4.2.8-1 to Fig.4.2.8-4 show that sliding contributed to the total horizontal displacement, especially in last cycle. The contribution of the sliding increased particularly when the horizontal crack running through the base line appeared.

The vertical cracks caused spalling of concrete cover at the compression corner. The spalling finally lead to concrete crushing at the lower corner, readings is beyond the range of the LVDT or the records are not the true value of sliding. It is evident as shown in Fig.4.2.8-1. However, it still provides a great deal of information in middle plastic stages. The malfunction occurred in LVDT to record sliding of specimen W3, which is not found during the test.

Sliding occurred in the monolithic walls and their precast counterparts are compared as shown in Table.4.2.8-1. The rate of sliding shows that monolithic prototypes experienced higher sliding deformation near the base than their precast counterparts did, which is about 20% of the total horizontal displacement in monolithic walls, whereas this rate in precast walls is about 10%. This is consistent with the experimental observations.

Fig. 4.2.8-1 Sliding Deformation of Specimen W1

Fig. 4.2.8-2 Sliding Deformation of Specimen W2

Fig. 4.2.8-3 Sliding Deformation of Specimen W4

Fig. 4.2.8-4 Sliding Deformation of Specimen W5

Table 4.2.8-1 Ratio of Sliding of Specimen to Total Displacement

Specimen	SN	Displacement	Sliding	S/D (%)
W1	604	33.6	5.76	17
W2	386	4.81	45.54	10
W4	750	9.3	58.65	20
W5	574	5.35	48.22	12

4.2.9 Deformations at Horizontal Connections

In precast specimen walls, the deformations occurred in the horizontal connections are important parameters affecting performance of these walls. Two horizontal LVDTs, two diagonal LVDTs and four vertical LVDTs, as shown in Fig.3.2.2-8, are used to monitor the horizontal deformations of the connection. The performance of the horizontal connection in specimen W2 is taken out as an example which is shown in Fig.4.2.9-1 to Fig.4.2.9-5. Fig.4.2.9-1 and Fig.4.2.9-2 show the records of L23 and L24 respectively, which are located at upper and lower walls panels near the horizontal connection. The difference between L23 and L24 is seen as the total horizontal deformation of the connection. The hysteresis loops of L23 and L24 are not symmetrical, which may be due to the horizontal expansion at these locations caused by internal forces. This will be explained in later sections. Fig.4.2.9-3 shows that the total horizontal deformation is almost symmetrical except in last cycle. The total horizontal deformation of the connection is decomposed into shear and flexural deformations as shown in Fig.4.2.9-4 and Fig.4.2.9-5. The flexural deformations exhibit stable behavior. And the shear deformation of the connection experiences some slippage which was caused by the friction behavior along the horizontal connection.

Fig. 4.2.9-1 Records of L23 versus Lateral Load of W2

Fig. 4.2.9-2 Records of L24 versus Lateral Load of W2

Fig. 4.2.9-3 Total horizontal Deformation at Horizontal Connection of W2

Fig. 4.2.9-4 Flexural Deformation at Horizontal Connection of W2

Fig. 4.2.9-5 Shear Deformation at Horizontal Connection of W2

4.2.10 Capacity of Energy Dissipation

There are different ways to evaluate the energy dissipation capacity of a structural member, which is a property that has long been recognized to be of importance with respect to the seismic performance of a structure. The total amount of hysteresis energy, that is, the cumulative area of the hysteresis loops when the structure fail, is an important parameter, but does not clearly indicate the level of deformation at which this energy is dissipated, and is also subjected to the uncertainties involving the definition of failure. Herein, when a 25% drop in strength (post-peak) occurs in a structure, it is considered to be failed.

Because strength of the walls varies, meaningful comparison can only be made between walls with similar strength. The energy dissipation also depends on the maximum displacement attained. In this research, all specimens have the same height, so they can be assessed directly. The total cumulative energy dissipation is dependent on the loading regime imposed, and in this study, can not be used on its own as a comparative measure for assessing performance. Because different loading regimes for these specimens were applied in this study, the total accumulated

energy is not suitable to be used to evaluate the capacity of energy dissipation of these specimens. Therefore, in order to compare the capacity of energy dissipation, the energy dissipation per displacement is presented in Fig.4.2.10-1 to Fig.4.2.10-5 and used as a tool to assess the performance of these walls; in each graph the virgin cycle and second cycle are given. The energy dissipation is drawn against the total horizontal displacement of the top beam, that is, at each displacement level the energy dissipation of the specimen in each cycle is divided by the horizontal displacement. And the energy dissipated in each cycle at every displacement level is calculated separately.

From the figures of energy dissipation of all these specimens, the observations show that in prior-yield stage a very small energy is dissipated. However, in the post-yield stage the virgin cycles dissipate more energy than the second cycles, especially in precast walls. The walls failed in flexure there is a sharper rate of dissipation of energy, which is almost constant up to failure. A drop of this rate indicates shear deterioration, which means significantly decrease in the total cumulative energy.

Fig.4.2.10-6 shows the comparison of energy dissipation per displacement of all specimens. a). Specimens with embedded I-Beams exhibits higher energy dissipation per displacement, which indicates that, with same loading history, the cumulative dissipated energy of specimens with embedded I-Beams has a higher capacity of energy dissipation than those walls with embedded channels. For monolithic walls, W4 has 13.1 percent higher capacity of energy dissipation at the peak point than W1 does, and for precast walls, W5 has 6.5 percent higher than W2 and 10.2 percent than W3. The major reason is the higher reinforcement ratio due to bigger sectional area of I-Beam than Channel, which caused higher strength of W4 than W1 at same displacement level. b). In pre-yield stage, the difference in energy dissipation between monolithic specimens and their relevant precast counterparts is about 8 percent for walls with embedded Channels, 10 percent for walls with embedded I-Beams. In post-yield stage, this difference for walls with embedded channels is 11.9 percent and is 18.8 percent for walls with embedded I-beam.

Fig. 4.2.10-1 Energy Dissipation per Displacement of Specimen W1

Fig. 4.2.10-2 Energy Dissipation per Displacement of Specimen W2

Fig. 4.2.10-3 Energy Dissipation per Displacement of Specimen W3

Fig. 4.2.10-4 Energy Dissipation per Displacement of Specimen W4

Fig. 4.2.10-5 Energy Dissipation per Displacement of Specimen W5

Fig. 4.2.10-6 Comparison of Energy Dissipation per Displacement of All Specimens

4.3 Theoretical Study

4.3.1 Shear Strength

Because the horizontal load was applied gradually in successive loading cycles, the strength of all specimens was evaluated analytically at any state of behavior, particularly at yield and ultimate state.

Eurocode 8 is probably the first document of regulatory character that separately treats the different modes of shear failure in RC walls subjected to seismic loading by providing a different design equation for each of the following modes:

- Diagonal tension failure
- Diagonal compression failure (web crushing)
- Sliding shear

Resistance V_{Rd3} against diagonal tension failure is calculated by superimposing a concrete contribution V_{cd} to a web reinforcement contribution V_{wd} , that is,

$$V_{Rd3} = V_{cd} + V_{wd} \quad (4.3.1-1)$$

$$V_{cd} = 0.06\sqrt{f_c} (1.2 + 40\rho_1) b_w z \quad (4.3.1-2)$$

$$V_{wd} = [\rho_h (\alpha_s - 0.3) + \rho_v (1.3 - \alpha_s)] f_{yd} b_w z \quad (4.3.1-3)$$

Where ρ_1 is ratio of tension reinforcement, b_w is thickness of web; z represents effective internal lever arm, about $0.8l_w$ (distance between the resultants of tensile and compression stresses at the section considered); ρ_h and ρ_v are ratios of horizontal and vertical web reinforcement, respectively; f_{yd} is design strength of this reinforcement; α_s is the shear-span ratio equal to $M/(Vl_w)$ where M is moment, V is shear force, l_w is wall length.

Equation (4.3.1-3) implies that for $\alpha_s = 1.3$, only horizontal web reinforcement is contributing to shear strength, whereas for $\alpha_s = 0.3$, only the vertical web

reinforcement is resisting shear; both types of web reinforcement are taken into account for $0.3 < \alpha_s < 1.3$.

Herein in order to compare theoretical and experimental average shear stress, formulas (4.3.1-1 ~ 3) are expressed by stress:

$$\tau_{Rd3} = \tau_{cd} + \tau_{wd} \quad (4.3.1-4)$$

$$\tau_{cd} = \frac{0.8V_{cd}}{b_w z} = 0.054 \sqrt{f_c} (1.2 + 40\rho_1) \quad (4.3.1-5)$$

$$\tau_{wd} = \frac{0.8V_{wd}}{b_w z} = 0.8[\rho_h(\alpha_s - 0.3) + \rho_v(1.3 - \alpha_s)]f_{yd} \quad (4.3.1-6)$$

Where τ_{Rd3} is diagonal tension stress, τ_{cd} is stress contribution by concrete to τ_{Rd3} , τ_{wd} is stress contribution by web reinforcement to τ_{Rd3} .

Eurocode 8 requires an explicit calculation of the resistance of walls against sliding shear failure; this resistance is assumed to be made up from the dowel action of vertical reinforcement, the shear resistance of cross-inclined bars, and the frictional resistance at the sliding plane. For squat walls ($h_w/l_w < 2.0$), it is suggested that in Eurocode 8 that at least half of the design shear at the base should be carried by diagonal bars (at higher levels, the contribution of these bars may be reduced to one-fourth the total shear). Diagonal bars must be properly anchored beyond the sliding plane and should cross all section of the wall within a distance of $0.5l_w$ or $0.5h_w$ (which is smaller) above the critical section.

Eurocode 8 gives formula to calculate the resistance of walls against sliding shear failure

$$V_{Rd,s} = V_{dd} + V_{id} + V_{fd} \quad (4.3.1-7)$$

$$V_{dd} = \min\{1.3 \sum A_{sj} (f_{cd} f_{yd})^{0.5}, 0.25 f_{yd} \sum A_{sj}\} \quad (4.3.1-8)$$

$$V_{id} = \sum A_{si} f_{yd} \cos \phi \quad (4.3.1-9)$$

$$V_{fd} = \min\{\mu_f [\sum A_{sj} f_{yd} + N_{sd}] \xi + M_{sd} / z, 0.25 f_{cd} \xi l_w b_w\} \quad (4.3.1-10)$$

Where V_{dd} is dowel action of vertical bars, V_{sd} is the shear resistance of diagonal bars (at angle ϕ) with assumption that diagonal bars yield simultaneously in tension and compression., and V_{fd} represents the friction resistance. $\sum A_{vj}$ is the sum of the areas of the vertical bars in the web and of purposely arranged bars in the boundary elements of the wall (that is, bars not required for flexural strength); $\sum A_{di}$ is the sum of the areas of all diagonal bars in both directions, $f_{cd} = f'_c / 1.5$ is the design strength of concrete; μ_f is the concrete-to-concrete friction coefficient under cyclic action (usually taken equal to 1.0 for rough concrete); $\xi = x / l_w$ is the normalized neutral axis depth; M_{sd} is the design moment, and N_{sd} is the design axial force at the potential sliding plane.

The diagonal compression failure mode (crushing of concrete struts formed in the web between inclined shear cracks) might be a problem in heavily reinforced walls, typical of multistory buildings in seismic areas, particularly when strong boundary elements of flanges are combined with relatively thin webs.

Formulas given by ACI 318-2002: (unit: lbs, psi) are the followings:

$$V_n = V_c + V_s \quad (4.3.1-11)$$

$$V_c = 3.3\sqrt{f'_c}hd + \frac{N_u d}{4l_w} \quad (4.3.1-12)$$

$$\text{or } V_c = \left[0.6\sqrt{f'_c} + \frac{l_w \left(1.25\sqrt{f'_c} + 0.2 \frac{N_u}{l_w h} \right)}{\frac{M_u}{V_u} - \frac{l_w}{2}} \right] hd \quad (4.3.1-13)$$

Horizontal reinforcement:

$$V_s = \frac{A_v f_y d}{s_2} \quad (4.3.1-13)$$

Vertical reinforcement:

$$\rho_n = 0.0025 + 0.5 \left(2.5 - \frac{h_w}{l_w} \right) (\rho_h - 0.0025) \quad (4.3.1-14)$$

The calculated $V_n=570\text{kN}$, therefore the average shear stress is 2.26MPa . And in ACI 318-02 there is no explicit formula to determine the shear strength to prevent sliding shear failure, but an indirect control, an upper bound of $0.81\sqrt{f'_c}\text{ MPa}$ ($10\sqrt{f'_c}\text{ psi}$) is provided to prevent this type of failure.

For specimens with embedded channels, W1, W2 and W3, comparing ultimate strength from test with results from code (ACI 318-2002) in which influence of ratio of height to width is not taken into account, it shows that the design strength of wall can effectively resist the ultimate lateral load, that is, flexural failure controls. However, horizontal shear failure occurred in W2 and W3.

Table 4.3.1-1 Comparison Of Calculated Results Basing On Codes And Experimental Result

	Eurocode 8						ACI 318-02		Experimental Average Shear stress	
specimen	τ_{cd}	τ_{wd}	τ_{Rd3}		$\tau_{Rd,s}$		τ_n		τ_{aver}	
	(MPa)	(MPa)	(MPa)	$\sqrt{f'_c}$	(MPa)	$\sqrt{f'_c}$	(MPa)	$\sqrt{f'_c}$	(MPa)	$\sqrt{f'_c}$
W1	0.54	2.72	3.26	0.55	2.02	0.34	2.26	0.38	2.23	0.37
W2	0.52	2.72	3.24	0.56	2.02	0.35	2.26	0.39	2.22	0.38
W3	0.53	2.72	3.25	0.56	2.02	0.35	2.26	0.39	2.27	0.39
W4	0.55	2.72	3.27	0.55	2.16	0.36	2.26	0.38	3.00	0.50
W5	0.53	2.72	3.25	0.56	2.16	0.37	2.26	0.39	2.94	0.50
W6	0.52	2.72	3.24	0.56	2.16	0.37	2.26	0.39	-	-

4.3.2 Theoretical Model (Macro Wall Model)

A. Introduction

This section is devoted to the development of a theoretical model of precast shear wall to predict the relationship of lateral load and displacement. To cater for the configuration of the precast walls and to capture the behavior of the horizontal connection which is composed of two vertical steel elements at boundary to resistance moment and one or two shear connectors in the central zone to take the

horizontal shear force. Macro model based on the nonlinear springs connected by rigid beams, as shown in Fig. 4.3.2-1, originally suggested by Kabeyasawa et al (1982), is the most suitable model. The two outer springs represent boundary columns while a horizontal spring at center works as web of wall to resist the lateral shear force, a vertical spring at center represents web of wall to take the vertical load and a rotation spring at center represents the contribution of web for the resistance of rotation.

To improve the accuracy of simulation on shear walls, Vulcano & Bertero (1987) suggest a vertical spring model by a set of springs, as seen in Fig. 4.3.2-2a, which simulate the physical behavior of cracking of concrete and yielding of reinforcement. The single spring on the top works as uncracked concrete while the parallel springs below model cracked concrete and steel, respectively. The steel spring follows a bilinear curve, and the concrete cracking-spring either seizes to act (crack state) or takes up action (closing of crack).

Vulcano et al (1988) replaced the rotation spring by several additional vertical springs to simulate the axial behavior of the wall web shown in Fig. 4.3.2-2b. This method was able to model the gradual yielding of the vertical reinforcement more smoothly, but it consists of more components and thus leads to a more complicated model.

Fig. 4.3.2-1 Original Macro Model by Kabeyasawa et al (1982)

Fig 4.3.2-2 Suggestion for Improved Macro Models

A simple model suggest by Linde (1993), as shown in Fig. 4.3.2-3, consists of three vertical springs, two at boundaries and one at center, and a horizontal spring at center. The rotation spring in Kabeyasawa model is omitted. A derivation of the properties for the outer vertical springs was done to satisfy the simulation the flexural behavior. The horizontal spring, modeling shear behavior, continued its function.

The formulas of the wall model were derived by Linde (1993) from a typical cantilever wall behavior, including elastic flexural behavior, nonlinear flexural behavior and elastic shear behavior. Different material models for the springs in

relevant suggested macro models will be derived and applied to capture the relationship between lateral force and displacement of a whole precast wall in subsequent sections.

Elastic Flexural Behavior

To derive the formulas, as shown in Fig. 4.3.2-4 and Fig 4.3.2-5, the kinematic relations of a simple real wall is compared with those its model. A real cantilever wall shown in Fig. 4.3.2-4a obeys the basic elastic theory on beams. Under pure flexure the model in Fig. 4.3.2-4b, would have to simulate the uniform curvature.

Fig. 4.3.2-4 Wall Rotation for Uniform Moment by Linde (1993)

From simple beam theory, as shown in Fig. 4.3.2-4a, following formula can be derived.

$$\theta = \frac{Mh}{EI} \quad (4.3.2-1)$$

$$\delta_h = \frac{Mh^2}{2EI} \quad (4.3.2-2)$$

Where h is height of the wall, l is width, E is Young's modulus, I is moment inertia.

And for Fig. 4.3.2-4b with $F_{sp} = \frac{M}{l}$, $\theta = \frac{2\delta_v}{l}$, and $\delta_v = \frac{F_{sp} h}{A_{sp} E} = \frac{Mh}{lA_{sp} E}$, we get

$$\theta = \frac{2Mh}{l^2 A_{sp} E} \quad (4.3.2-3)$$

$$\delta_h = \theta h_c = \frac{M h h_c}{l^2 A_{sp} E} \quad (4.3.2-4)$$

Where A_{sp} is the spring area, h_c is distance from rotation center to the top of the wall,

By setting (4.3.2-1) equal to (4.3.2-3), and (4.3.2-2) equal to (4.3.2-4), model properties is gotten to represent the outer spring area and relative center of wall rotation, respectively.

$$A_{sp} = \frac{2I}{l^2} \quad (4.3.2-5)$$

$$h_c = \frac{h}{2} \quad (4.3.2-6)$$

The elastic stiffness of the outer spring is given by

$$K_{sp,e} = \frac{A_{sp} E}{h} \quad (4.3.2-7)$$

Fig. 4.3.2-5 Wall Rotation with Moment Gradient by Linde (1993)

Normally in shear wall structures, the lateral force acts on each story and causes bending moment gradient along the wall as shown in Fig. 4.3.2-5. As done above, the properties of model were derived through the equation 4.3.2-5 to 4.3.2-11.

For real elastic wall shown in Fig. 4.3.2-5a,

$$\theta = \frac{Vh}{2EI} \quad (4.3.2-8)$$

$$\delta_h = \frac{Vh^3}{3EI} \quad (4.3.2-9)$$

For macro model shown in Fig. 4.3.2-5b

$$F_{sp} = \frac{Vh_c}{l} = \frac{Vh}{2l} \quad (4.3.2-19)$$

$$\delta_v = \frac{F_{sp} h}{A_{sp} E} = \frac{Vh^2 l}{4EI} \quad (4.3.2-11)$$

$$\theta = \frac{2\delta_v}{l} = \frac{Vh^2}{2EI} \quad (4.3.2-12)$$

$$\delta_h = \theta h_c = \frac{Vh^3}{4EI} \quad (4.3.2-13)$$

The deflection given by elastic theory (equation 4.3.2-9) differs from that from model. This deficiency can be reduced by discretization using several elements verified by Linde (1988).

And if keep h_c as parameter in the formula, we can get,

$$\delta_v = \frac{F_{sp} h}{A_{sp} E} = \frac{Vh h_c}{l A_{sp} E} \quad (4.3.2-14)$$

$$\theta = \frac{2\delta_v}{l} = \frac{2Vh h_c}{l^2 A_{sp} E} \quad (4.3.2-15)$$

$$\delta_h = \theta h_c = \frac{2Vh h_c^2}{l^2 A_{sp} E} \quad (4.3.2-16)$$

Through equating (4.3.2-8) to (4.3.2-15) and (4.3.2-9) to (4.3.2-16), we can get,

$$A_{sp} = \frac{8I}{3l^2} \quad (4.3.2-17)$$

$$h_c = \frac{2h}{3} \quad (4.3.2-18)$$

Comparing equation (4.3.2-18) with (4.3.2-6), the center of rotation under only shear force is lower than that in wall under pure bending moment. General case is that at a wall section bending moment and shear force exist simultaneously. To cater for the real wall, in this research, a combination of the two situations is applied according to the proportion of the moment and shear force.

Elastic Shear Behavior

A horizontal spring is used to simulate the shear behavior as shown in Fig. 4.3.2-6. From simple elastic theory, the horizontal deflection caused by the shear is

$$\delta_{he} = \frac{\kappa V h}{G A_w} \quad (4.3.2-19)$$

$$\kappa = \frac{0.75(1+u)[1-u^2(1-v)]}{1-u^3(1-v)} \quad (4.3.2-20)$$

κ is a empirical factor suggested Tommii and Osaki (1955) and definition of factors u and v is shown in Fig. 4.3.2-7.

Fig. 4.3.2-6 Wall Shear Behavior by Linde (1993)

Fig. 4.3.2-7 Wall Shear Behavior by Linde (1993)

B. Suggested Model for Precast Walls

This study is intended to develop simple and clear formulations of the macro model for precast shear walls without vertical load. Because no axial load was applied in this study and purpose of development of the model is to predict the relationship between lateral load and displacement, the property of the vertical spring at center is not a critical factor in this research and has negligible influence on the accuracy of prediction. The suggested model is shown in Fig. 4.3.2-8, with two vertical outer springs, one dummy vertical spring and one horizontal spring. The idea behind the arrangement is to achieve simplicity by using as few springs as possible. Because the beams connecting the nonlinear springs are flexurally rigid, the kinematic possibilities are essentially the same as for models with more complicated spring configurations, and thereby this model is able to provide an efficient result, with the nonlinear behavior derived properly.

Fig 4.3.2-8 Suggested Macro Models

Because of the totally different features between the horizontal connection and RC wall panels in the precast walls, the whole precast wall will be simulated by an assembly of the macro models as shown in Fig 4.3.2-9. The suggested model of precast wall consists of three components, one macro model of connection (MMC) and two macro model of wall (MMW). The concept is listed out as following.

In precast walls the horizontal connections were composed of two boundary steel sections and one or two shear connectors at center of the wall web. At the

horizontal connection, the boundary steel members connected by bolts form flexural a moment resistant system as those in macro model while the shear connectors at center of the horizontal works the shear spring in the macro model. These features are consistent with the configuration of the macros model.

For the upper and lower wall panels with concentrated steel at boundaries the major configurations of the macro model were satisfied.

Fig 4.3.2-9 Suggested Macro Models for Precast Shear Walls W2,3,5,6

a) Material Models for MMC

Material model for outer springs

At the horizontal connection level, moment and shear force exist simultaneously, as shown in Fig. 4.3.2-10, which caused rotations as shown in Fig. 4.3.2-11. Because of the existence of the horizontal connection, the tension and compression forces caused by moment worked like tension out and push in action

of rebars. To simplify the bond stress distribution, an equivalent bond stress distribution is applied in this research, as shown in Fig. 4.3.2-12. An equivalent of rebar is utilized with same sectional area and perimeter.

For example, in the horizontal connection of W2, two channel were used with total sectional area $856 \times 2 = 1712 \text{ mm}^2$ and perimeter $(76 + 38 \times 2) \times 2 \times 2 = 608 \text{ mm}$

The equivalent reinforcement is 17.2 nos diameter 11.26mm rebars as shown Fig. 4.3.2-13.

Bond stress of plain reinforcement has studied by Feldman and Bartlett (2005, 2007) and found that The average bond strength was 0.98 MPa for as-received bars, and increased by 124% to 2.2 MPa for bars sandblasted to simulate realistic surface roughness. Mosley (1999) proposed for plain reinforcement is $0.28\sqrt{f_{cu}}$ for tension and $0.35\sqrt{f_{cu}}$ for compression, which will be applied in this study.

$$M = \frac{Vh_1}{l} \quad (4.3.2-21)$$

$$F_{sp} = \frac{M}{l} \quad (4.3.2-22)$$

$$f_{sp} = \frac{F_{sp}}{A_{sp}} \quad (4.3.2-23)$$

$$l_d = \frac{f_{sp} d}{4u_b} \quad (4.3.2-24)$$

Where F_{sp} is force in outer spring, u_b is the bond strength, f_{sp} is stress in spring, l action length of bond.

When spring in elastic stage,

$$\delta_{v,t} = 2\left(\frac{1}{2}\varepsilon_{sy}l_e\right) = \frac{f_{sp}}{E}l_e \quad (4.3.2-25)$$

When spring goes into plastic stage,

$$\delta_{v,t} = 2\left[\frac{1}{2}\varepsilon_{sy}l_e + \varepsilon_{sy}l_p + \frac{1}{3}(\varepsilon_{sp} - \varepsilon_{sy})l_p\right] \quad (4.3.2-26)$$

Where ε_{sy} and ε_{sp} strain in yielding and plastic, l_e is elastic action length of bond, l_p is plastic action length of bond.

Because the connection is bolt connection, the tolerance, t_t , in between bolts and bolt holes should be included.

$$\theta = \frac{\delta_{v,t} + t_t}{l/2} \tag{4.3.2-27}$$

The horizontal displacement at top beam caused by the rotation of the horizontal connection is

$$\delta_{hc1} = \theta h_1 = \frac{\delta_{v,t} + t_t}{l/2} h_1 \tag{4.3.2-28}$$

Fig 4.3.2-10 Load Status in the Horizontal Connection

Fig 4.3.2-11 Typical Deformation of the Horizontal Connection

Fig 4.3.2-12 Equivalent Stress Distribution

Fig 4.3.2-13 Material Model for Outer Springs

Material model for shear springs

As shown in the Fig. 4.3.2-10, the shear force is applied at center of the model. To determine the horizontal displacement caused by shear force is to determine the property of the shear connectors in the horizontal connection. The connectors were mounted to the wall panels by embedded reinforcement. From Fig. 4.3.2-14, the shear force is resisted by the tension and compression reinforcement. Here use $0.5\sqrt{f_{cu}}$ for reinforcement in tension and $0.63\sqrt{f_{cu}}$ for reinforcement under compression as the working bond stress, suggested by Mosley (1999).

From Fig. 4.3.2-14, we can get

$$V = (T + C) \cos \alpha \quad (4.3.2-29)$$

$$\frac{C}{T} = \frac{0.63}{0.5} = 1.26 \quad (4.3.2-30)$$

$$T = \frac{V}{2.26 \cos \alpha} \quad (4.3.2-31)$$

$$f_d = \frac{T}{A_d} \quad (4.3.2-32)$$

$$l_{d,sp} = \frac{f_d d}{4u_b} \quad (4.3.2-33)$$

Where α is the inclined angle of diagonal reinforcement, f_d is stress in diagonal reinforcement, $l_{d,sp}$ is the action length of diagonal reinforcement.

When spring in elastic stage,

$$\delta_{d,t} = 2\left(\frac{1}{2} \varepsilon_{sy} l_e\right) = 2\left(\frac{f_d}{2E} l_e\right) \quad (4.3.2-34)$$

When spring goes into plastic stage,

$$\delta_{d,t} = 2\left[\frac{1}{2} \varepsilon_{sy} l_e + \varepsilon_{sy} l_p + \frac{1}{3} (\varepsilon_{sp} - \varepsilon_{sy}) l_p\right] \quad (4.3.2-35)$$

$$\delta_{hc2} = 2\left(\frac{1}{2} \varepsilon_d l_d\right) \cos \alpha = \frac{f_d}{E} l_d \cos \alpha + t_l \quad (4.3.2-36)$$

Fig 4.3.2-14 Forces in Diagonal Reinforcement in the Shear Connection

Therefore, the total lateral deflection caused by deformation of the horizontal connection is

$$\delta_{hc} = \delta_{hc1} + \delta_{hc2} \quad (4.3.2-37)$$

Where δ_{hc1} is the horizontal deformation by moment, δ_{hc1} is the horizontal deformation by shear and tolerance

b) Nonlinear Material Models for MMW

From Fig. 4.3.2-9, the load on the MMW can be gotten as shown in Fig. 4.3.2-15.

Fig 4.3.2-15 Loads on MMW

Material model for outer springs

The flexural behavior of the MMW, including deformation by moment and by shear, is shown in Fig 4.3.2-16. To determine of the model properties beyond the linear elastic region it is necessary to study the physical behavior of the cross section of a real wall. The steel reinforcement and embedded steel sections and concrete worked following their own material constitutive laws, respectively. In present research, an idealized trilinear material model shown in Fig 4.3.2-17a is applied for steel. And the concrete model by Kent and Park (1971) as shown Fig 4.3.2-18 is used.

$$f_c = f'_c \left[\frac{2\varepsilon_c}{0.002} - \left(\frac{\varepsilon_c}{0.002} \right) \right] \text{ for } \varepsilon_c \leq 0.002 \quad (4.3.2-38)$$

$$f_c = f'_c [1 - Z(\varepsilon_c - 0.002)] \text{ for } \varepsilon_c \geq 0.002 \quad (4.3.2-39)$$

Where

$$Z = \frac{0.5}{\varepsilon_{50u} - \varepsilon_{50h} - 0.002} \quad (4.3.2-40)$$

$$\varepsilon_{50u} = \frac{3 + 0.002 f'_c}{f'_c - 1000} \quad (4.3.2-41)$$

$$\varepsilon_{50h} = 0.75 \rho_s \sqrt{\frac{b_h}{s_h}} \quad (4.3.2-42)$$

Where ρ_s is the confinement ratio, b_h is the width of the hoops, and s_h is the spacing of the hoops, and f'_c is the concrete cylinder strength in psi (1 psi=0.00689 N/mm²), ε_{50u} and ε_{50h} represents the strain at 50% strength on the descending branches of the unconfined and confined concrete.

In present study, the stage before cracking of concrete is not considered, because this will not affect the service limit state of the common precast shear walls. The moment for cracking can be gotten in RC design codes, and then the cracking lateral force can be derived through dividing the cracking moment by lever arm.

Fig 4.3.2-16 Flexural Behavior of MMW

Fig 4.3.2-17 Property of Flexural Springs

Fig 4.3.2-18 Concrete Material Model by Kent & Park (1971)

To simplify the model, a simple calculation model for the elastic stiffness in compression is suggested as shown in Fig 4.3.2-17b and the concrete under compression follow the material model in Fig 4.3.2-18. When the spring in tension, the concrete lose its strength after flexural cracked appears, only steel takes effect in the model without considering the tension stiffening effect of surrounding concrete between cracks.

Material model for shear springs

As observed in the experiment, the major shear deformation occurred in the lower wall panel. After diagonal cracking appeared, the lower panel started to experience plastic shear deformation. To simplify the model, an empirically-based ratio α_s of cracked to uncracked shear stiffness Wight (1984) is applied in this research.

$$\alpha_s = \frac{0.46\rho_w}{f'_c} + 0.14 \quad (4.3.2-43)$$

Where ρ_w is the horizontal reinforcement ratio. The parameter α_s , normally ranging from 0.15 to 0.20, was derived for wall cross sections of different geometry ranging from rectangular wall to wall with pronounced protruding boundary elements. Therefore the equation (4.3.2-19) to be

$$\delta_{hp} = \frac{\kappa V h}{\alpha_s G A_w} \quad (4.3.2-44)$$

C. Results Comparison

Because the cracking moment is not a critical control in service limit state, and also to simplify the prediction, present prediction started from lateral load higher than cracking load onwards. In the course of simulation, a simple Excel sheet is written to facilitate the prediction, which incorporates structural models of MMC and MMW, material models for steel and concrete. Random loading increments were input into the calculation sheet and output was drawn out versus experimental data as shown in Fig. 4.3.2-19 for precast walls W2 and W3, in Fig. 4.3.2-20 for precast walls W5 and W6. The comparisons show reasonable good agreement with experimental measurements in term of yielding strength, yielding displacement and ultimate strength. Accurately capture the yielding strength and displacement means the model accurately predicted the secant stiffness when structure reached yielding point, which is very important in design application. Good prediction on ultimate load indicates that the suggested model can be used when checking structure design redundancy for the precast walls with horizontal connection. This verified that the

suggested simple model could be applied in actual design of precast wall with horizontal connections.

Fig 4.3.2-19 Comparison of Prediction vs. Experimental Data of W2,3

Fig 4.3.2-20 Comparison of Prediction vs. Experimental Data of W5,6

D. Parametric Study

In course of experimental data analysis and simulation by the suggested model, it is interesting to compare result under different parameters, such as, bolt type and location of the horizontal connection. In present study, tolerance between bolt and bolt hole is 1 mm, which affected the behavior of the connection and thus influences the performance of the whole precast wall, including yield displacement, stiffness, ultimate displacement. High strength friction bolt is alternative to control the behavior of the connection, in which no tolerance is allowed. Location of the horizontal connection is another factor may affect the whole behavior of the wall, for example, the connection at bottom of the wall.

It can be seen from Fig. 4.3.2-21, the model with high strength friction bolts shows a reduction in the yielding displacement with the same yielding strength, which means higher stiffness for precast wall with high strength friction bolts.

To study effect of the location of the horizontal connection on the behavior of the precast wall, a typical model is shown Fig. 4.3.2-22. In the model only one horizontal connection is placed at bottom of the wall. The prediction result on this model is compared with experimental data W2 and W3 as shown in Fig. 4.3.2-23. The model shows lower stiffness, higher yielding displacement and lower ultimate strength. One reason is the less cross section area because when connection at bottom, no reinforcement surrounding the steel channels is added, which means less force can be taken by the spring in MMC. Tolerance in the connection causes additional rotation for the connection, and the additional horizontal displacement equals to this rotation times whole height of the wall. This is another reason.

As a results parametric study on the type of bolt and location of the connection the followings is recommended.

1. High strength friction bolts or weld is preferred in precast connection, which not only reduces the displacement from flexural deformation but also prevent the sliding along the connection due to tolerance.

2. Stronger steel section or section with bigger cross sectional area should be used in the connection to cater for the decrease in the steel section area where no reinforcement can be placed.

Fig 4.3.2-21 Comparison of Prediction with HSF Bolts vs. Experimental Data of W2,3

Fig 4.3.2-22 Parametric Study Model for Connection at Bottom

Fig 4.3.2-23 Comparison of Prediction with Connection at Bottom
vs. Experimental Data of W2,3

E. Conclusions

1. In this section, a simple model based on analysis on the features of the precast walls, which is an assembly of macro models, including macro model of connection (MMC) and macro model of wall (MMW), was suggested with corresponding constitutive laws.
2. The suggested model did a reasonable good prediction for precast walls on load-displacement relationship, including yielding displacement and ultimate load.
3. A parametric study was done on the type of bolt and location of the horizontal connection. High strength friction bolts improved the stiffness of the precast wall and thus reduce the structure deformation. The location of horizontal connection also affected the behavior of the precast walls and increased the lateral deformation. Stronger section or section with bigger area should be used in the connection.

4.3.3 Curvature Ductility

There are various types of ductility which influence behavior. Herein, only strain ductility proposed by Tasnimi (2000) and Chen (2005) is used to evaluate the behavior of specimens. Therefore assuming linear strain distribution across the section under consideration and perfect bond between concrete and reinforcement the value of ductility in all post-yield states could be calculated. Curvature at any post-yield state is the ratio of concrete compressive strain at any post-yield state to depth of neutral axis at corresponded state. On the other hand, curvature at the yield state is the ratio of yield strain of steel reinforcement to the depth of concrete in tension.

$$\mu_p = \frac{\phi_p}{\phi_y} = \frac{\varepsilon_{ci}(d-c)}{\varepsilon_{sy}c} \quad (4.3.3-1)$$

Where μ_p is curvature ductility, ϕ_p post-yield curvature, ϕ_y yield curvature, d length of lever arm, c depth of neutral axis, ε_{sy} is yielding strain of steel reinforcement in tension.

It is assumed that Δ_y at the top of wall is corresponded to the first yield of the section and the displacement at ultimate state Δ_u corresponded to the ultimate load.

The prediction of ultimate curvature ϕ_u was carried out using the ACI method and value based on test results.

Assuming the strain of first yield of reinforcement in the tension side of the wall section, and concrete compressive strain at extreme equal to 0.003, the ductility of the section, according to the reinforcement concrete design described in ACI 318-05, can be evaluated by:

$$\mu_p = \frac{\phi_u}{\phi_y} = \frac{\beta_1 \varepsilon_{cu} (1-k)}{\alpha \varepsilon_{sy}} \quad (4.3.3-2)$$

Where β_1 shall be taken as 0.85 for concrete strength up to and including 30MPa. For concrete strengths above 30MPa, β_1 shall be reduced continuously at a rate of 0.008 for each 1MPa of strength in excess of 30MPa. However, β_1 shall not be taken less than 0.65 (ACI 318-05)

The value of k is given as:

$$k = \left[(\rho + \rho')^2 m^2 + 2(\rho + \rho') \frac{d'}{d} m \right]^{1/2} - (\rho + \rho') m \quad (4.3.3-3)$$

$$\alpha = \frac{A_{sk} f_{yk} - A_{sj} f_{yj}}{0.85 b f'_c} \quad (4.3.3-4)$$

Where ε_{cu} and ε_{sy} are the concrete compressive strain at ultimate state and the yield strain of reinforcement. The modular ratio is represented by m and b is the width of the wall. ρ and ρ' are the tensile and compressive steel percentage, respectively.

Compression and tension forces at yield state are represented by $A_y f_y$ and $A_{sk} f_{yk}$, d' and d are distance of center of reinforcement in compression and tension to extreme compression fiber.

Table 4.3.3-1 Comparison Curvature Ductility Based On ACI And Experimental Result

Specimen	Ductility on ACI	Calculate curvature ductility based on test results			Ratio of calculated ductility to ACI
	μ_p	$\phi_y 10^{-6}$	$\phi_p 10^{-6}$	μ_p	
W1	6.44 (8.96)*	6.67	36	5.4	0.84 (0.6)*
W4	6.45 (8.42)*	8.03	49.4	6.15	0.95 (0.73)*

* Represents curvature ductility basing on code-specified nominal strength of steel and design strength of concrete.

Curvature of ductility, which indirectly represents flexural deformation of specimen, is calculated basing on rotation of plastic hinge zone in monolithic reinforcement concrete shear walls. From table, it is shown that for monolithic walls, W1 and W4, curvature ductility calculated basing on tested materials exhibits high agreement. This agrees with curvature figures shown in chapter 4.2.7, in which main curvature focuses in lower $\frac{1}{4}$ height of walls and curvature in upper $\frac{3}{4}$ height is negligible. However, curvatures in precast walls, W2, W3 and W5 are different from those in monolithic walls. It can be seen from table that curvature ductility based on code-specified nominal strength of steel and design strength of concrete is higher than that based on tested material and that calculated from experimental results. It hints that yield strength of flexural reinforcement exhibits reverse effect on curvature ductility of walls.

In order to evaluate curvature occurred at the horizontal connection during load course, the yielding curvature in the connection is calculated when reinforcement at bottom of the walls yield, and the ultimate curvature is based on the ultimate displacement of walls. Results from the suggested macro model and

experiment are compared in Table 4.3.3-2. The experimental data shows higher rotation than the model prediction.

4.4 summary

In this chapter, experimental results have been analyzed to qualify and quantify seismic performance of these specimens. The author also suggested a macro model which is an assembly of the macros model with different material models to predict the lateral-displacement relations for those precast walls.

1. Monolithic prototypes, W1 and W4, exhibit higher stable in plastic stage than those precast walls, W2, W3, W5 and W6 do. In monolithic walls, ratio of flexural deformation to total horizontal displacement is higher than that in precast walls.
2. Pinching effect is severer in precast walls due to higher shear deformation to total deformation. Through trends of ratios of flexural and shear deformation to total deformation, failure mode can be accurately predicted.
3. Average energy dissipation shows that the monolithic prototypes experience higher energy dissipation than their precast counterparts before ductility reach 2.
4. A macro model is suggested by the author, which produces a reasonable good prediction on the lateral load-displacement relation for precast walls, including stiffness, yielding displacement and ultimate strength.
5. Parametric study based on the suggested macro model has been done, including the type of bolt and location of the connection. High strength friction is preferred for application connection with bolts. This means similar stiffness as that of monolithic wall can be achieve with high strength friction bolts. And stronger section or section with bigger section area is

should be used to ensure strength and decrease deformation.

Table 4.3.3-2 Comparison of Theoretical and Experimental Curvature

Specimen	Theoretical calculation										experimental			Ratio of experiment al ductility to theoretical
	A_c (mm ²)	E_s (N/mm ²)	c (mm)	ϵ_m	P_t (kN)	l_t (mm)	h_c (mm)	ϕ_s (10 ⁻⁶)	ϕ_e (10 ⁻⁶)	μ_t	ϕ_s (10 ⁻⁶)	ϕ_e (10 ⁻⁶)	μ_t	
W2	1446.8	192460.35	120	0.003	499.88	1598	1390	3.1	4.7	1.5	4	6.9	1.73	1.15
W3	1446.8	192460.35	120	0.003	499.88	1598	1390	3.1	4.7	1.5	3.7	7.1	1.91	1.27
W5	2039.8	177279.7	140	0.003	567.46	1598	1390	2.17	4.01	1.85	3	6.02	2.01	1.09

CHAPTER 5

DISCUSSION OF TEST RESULTS

Strut-and-tie models have become increasingly popular tools for the design of reinforced concrete structures. They can be especially useful in regions of discontinuity. As a valuable tool to understand flow of internal load forces, strut-and-tie models are used to analyze behavior of these walls in this chapter.

5.1 Behavior of Specimen W1

Specimen W1 is a monolithic wall with embedded channels at two ends acting as prototype of specimen W2 and W3. This wall finally exhibited flexural failure with concrete crushing at right lower corner and breaking of weld of horizontal reinforcement. Characteristics of behavior of specimen W1 were revealed by the strain gauges mounted on the steel reinforcing bars and embedded steel sections and cracking patterns. Strut-and-tie models are built to give details of performance. As the results will verify, the model is a valid tool to display the behavior of the specimen.

5.1.1 Strain History of the Gauges

A. Strain History of Vertical Bars

Strain histories of vertical bars on both sides of wall are shown in Fig.5.1.1-1 to Fig.5.1.1-10. Vertical bars at ends of the walls were broken during the loading course with sound which can be seen from the strain histories of these vertical bars. The vertical bars with three strain gauges are selected to illustrate the main characteristics. And yield strain is also drawn in these figures and is used as

standard to evaluate states of these bars.

- a. In elastic stages, the gauges recorded negative values. However, in plastic stages, no negative readings are found except some broken the gauges. This is due to the permanent deformations at the locations of the gauges which came from cracks. The cracks crossing these bars open when in tension and plastic deformation occurred in these bars. When load reversed, the cracks could not close perfectly.
- b. All the vertical reinforcement yielded during test which can be seen from the strain recorded by the strain gauge named SVL series when lateral load reach peak values in plastic stages.
- c. Strain histories of SVM series at peak loads show plastic deformation of the bars at this height level. It indicates the plastic deformation of these bar extend to half height of the wall except 4 bars, such as SVM5, SVM6, NVM5 and NVM6, in middle of the wall. This means that there is an elastic core at SVM height level.
- d. Strain histories SVH series located at top of wall indicate that no plastic deformation occurred in any vertical bar at this height level since the reading of strain at peak loads is in elastic range.

Fig. 5.1.1-1 Strain History of Vertical Bar SV1

Fig. 5.1.1-2 Strain History of Vertical Bar NV2

Fig. 5.1.1-3 Strain History of Vertical Bar SV3

Fig. 5.1.1-4 Strain History of Vertical Bar SV4

Fig. 5.1.1-5 Strain History of Vertical Bar SV5

Fig. 5.1.1-6 Strain History of Vertical Bar SV10

B. Strain Profile of Vertical Bar at Base Line

The strain profiles of vertical bars along the baseline when load is at peak values, positive and negative, are shown in Fig.5.1.1-7 to Fig.5.1.1-8. The strain profiles on N-side in different cycles are presented. It can be seen from these figures that in early cycles the strain profiles approximately comply with the assumption of plane section.

The intersections of the strain profiles and the horizontal axis are shown in these figures, which can be approximately seen as the neutral axis. The compression depth is consistent with the calculation in specimen design. It showed that large vertical deformation occurred in the field about 1/3 width of the wall from the extreme tension fiber.

Fig. 5.1.1-7 Strain Profile of Vertical Bar at the Base Line of Specimen W1 for Negative Load

Fig. 5.1.1-8 Strain Profile of Vertical Bar at the Base Line of Specimen W1 for Positive Load

C. Strain Profiles along the Vertical Bars

Fig.5.1.1-9 to Fig.5.1.1-14 show the strain profiles along the vertical bars in different cycles. The strain profiles when load is at peak values, positive and negative, are given. Strain profiles of vertical bar on S-side SV1 are presented in Fig.5.1.1-9 and Fig.5.1.1-10 for negative and positive load, respectively. Strain tends to decrease along height of the wall. Under negative load, the strain at top is very small, whereas under peak positive load this strain tends to a higher value. The bar NV4 also shows similar tendency of SV1 as shown in shown in Fig.5.1.1-11 and Fig.5.1.1-12.

The strain profiles along NV6, which is one of bars nearest to center line of the wall, are shown in Fig.5.1.1-13 and Fig.5.1.1-14 in different cycles when lateral load reach peak values, negative and positive. No negative strain value appears in NV4 and NV6. The peak value of strain on NV6 is much less than those in other bars. Strain of NV6 at top of wall tends to zero, which is consistent with observed cracking patterns of this wall as shown in Fig.4.2.4-1 when ductility reaches 2. Tension force in this vertical bar is transferred to concrete by bond; as a result, this force is transmitted to the top beam by the concrete in tension.

Fig.5.1.1-9 Strain Profiles of along Vertical Bar SV1 for Negative Load

Fig.5.1.1-10 Strain Profiles of along Vertical Bar SV1 for Positive Load

Fig.5.1.1-11 Strain Profiles of along Vertical Bar NV4 for Negative Load

Fig.5.1.1-12 Strain Profiles of along Vertical Bar NV4 for Positive Load

Fig.5.1.1-13 Strain Profiles of along Vertical Bar NV6 for Negative Load

Fig.5.1.1-14 Strain Profiles of along Vertical Bar NV6 for Positive Load

D. Strain Profile of Steel Sections

The strain profiles of all Channels at different height are shown in Fig.5.1.1-15 to Fig.5.1.1-22. The layout of strain gauges on steel channels is shown in Fig.3.2.2-4. Strain gauges pasted on inner channels on S-side include series of SS1 and SS2, while the other strain gauges placed on outer channels on N-side are divided into series of NN1 and NN2. In order to catch characteristics of strain of channels, more strain gauges are placed on each channel than those on vertical rebars. In the first cycle, the strain profile exhibits linear tendency from base to top, which is consistent with the moment on each section of wall. With increasing displacement levels, the deformation of channels near the base beam increases much faster than upper part and the strain profiles undergo nonlinear trend. The main deformation occurs in lowest part, which means this part experienced much vertical elongation. These characters are similar to those of vertical rebars shown in previous section.

As shown in Fig.5.1.1-15 and Fig.5.1.1-16, the strain profiles of SS1 at the location about 800mm from base beam from cycle 3 in negative and positive loading courses. However, this sudden increase in strain does not occur in Channel NN1 as shown in Fig.5.1.1-19 and Fig.5.1.1-20, which is at same end of the wall. And similar difference happens between SS2 as shown in Fig.5.1.1-17 and Fig.5.1.1-18, in which the sudden increase of strain locate at 1200mm from base beam, and NN2 as shown in Fig.5.1.1-21 and Fig.5.1.1-22. All these sudden increase in strains occurred at these location means regions may experienced a crack near the gauge.

Fig. 5.1.1-15 Strain Profile of Channel SS1 for Negative Load

Fig. 5.1.1-16 Strain Profile of Channel SS1 for Positive Load

Fig. 5.1.1-17 Strain Profile of Channel SS2 for Negative Load

Fig. 5.1.1-18 Strain Profile of Channel SS2 for Positive Load

Fig. 5.1.1-19 Strain Profile of Channel NN1 for Negative Load

Fig. 5.1.1-20 Strain Profile of Channel NN1 for Positive Load

Fig. 5.1.1-21 Strain Profile of Channel NN2 for Negative Load

Fig. 5.1.1-22 Strain Profile of Channel NN2 for Positive Load

E. Strain History of Diagonal Bars

The histories of strain in diagonal bars near the base are shown in Fig.5.1.1-23 to Fig.5.1.1-26. From Fig.5.1.1-23 and Fig.5.1.1-24, when lateral load is negative,

there is big jump in strain of ND1 and ND2. This means that diagonal cracks opened which cross the strain gauges mounted on the diagonal reinforcement. This is consistent with experimental observation. From reading of strain of strain gauges on diagonal bars, both positive and negative values are experienced in the cyclic load course, which means that whether in compression or in tension, diagonal bars play their roles in resisting lateral load.

As shown in Fig.5.1.1-23 to Fig.5.1.1-26, after ductility reaches 4, the increase of maximum strain in all diagonal reinforcement in tension when lateral load reaches peak value at every displacement level, positive and negative, is not considerable while the total displacement of the specimen increases, which means that there is not much increment of strain due to plastic deformation and diagonal rebars did not reach their ultimate strength. There were cracks passing through or near the gauge. This could be seen from apparent strain increment, from about $500 \mu\epsilon$ to $1000 \mu\epsilon$. The final maximum strain was about $3500 \mu\epsilon$, which was in the measure range and did fail the gauge. This indicated that the cracks were not so wide, which was in agreement with the observed cracking patterns. And it is also that these bars yield when they in tension. The contribution to lateral load can be evaluated.

Fig.5.1.1-23 History of Strain in Diagonal Bar ND1

Fig.5.1.1-24 History of Strain in Diagonal Bar SD2

Fig.5.1.1-25 History of Strain in Diagonal Bar SD3

Fig.5.1.1-26 History of Strain in Diagonal Bar SD4

F. Strain History of Horizontal Bars

Strain histories of all horizontal reinforcement are shown in Fig.5.1.1-27 to Fig.5.1.1-38 from top to base in sequence.

1. During elastic cycles 1 to 3 the strain is negligible. From cycle 4 to ductility 2, considerable increase can be observed in reading of these strain gauges.
2. It can be seen in these figures that a pronounced increase in strain with less increase in lateral load occurred when a diagonal crack passing through the strain gauges.
3. In most cases during plastic cycles, the permanent residual strain after reversed load applied was about $500\ \mu\epsilon$ to $800\ \mu\epsilon$ for most horizontal strain gauges. This means that these diagonal cracks may not have closed completely, even though load reversed.
4. The characteristics of distribution of strain of horizontal reinforcement along height also are obtained from these figures. From NH2 to SH5, the maximum strain at peak load is about $1500\ \mu\epsilon$, strain of NH6 is about $1900\ \mu\epsilon$. SH7 and NH8 experience higher strain more than $2700\ \mu\epsilon$ at peak load, which indicates that they yielded under peak load. In the region with diagonal bars, the maximum strain decreased at peak load from top end of diagonal bars to base line, which means that in the lower part of the wall, the diagonal reinforcement partook of lateral load and decreased strain in horizontal reinforcement. From peak readings of these strain gauges the contribution of the horizontal reinforcement to shear can be evaluated.

Fig.5.1.1-27 Strain History of Horizontal Bar SH1

Fig.5.1.1-28 Strain History of Horizontal Bar HN2

Fig.5.1.1-29 Strain History of Horizontal Bar SH3

Fig.5.1.1-30 Strain History of Horizontal Bar NH4

Fig.5.1.1-31 Strain History of Horizontal Bar SH5

Fig.5.1.1-32 Strain History of Horizontal Bar NH6

Fig.5.1.1-33 Strain History of Horizontal Bar SH7

Fig.5.1.1-34 Strain History of Horizontal Bar NH8

Fig.5.1.1-35 Strain History of Horizontal Bar SH9

Fig.5.1.1-36 Strain History of Horizontal Bar NH10

Fig.5.1.1-37 Strain History of Horizontal Bar NH11

Fig.5.1.1-38 Strain History of Horizontal Bar SH12

5.1.2 Load Transfer

The mechanism of shear transfer consists of aggregate interlock along crack interface, dowel action of chord reinforcement, shear transfer by concrete in the flexural compression zones and tensile strength of uncracked concrete.

As shown in Fig.4.2.4-1, there was a wide horizontal crack passing across the base line of wall, which appeared at DF equal to 5. In late plastic cycles, if in compression, the crack which previously opened in tension almost closed. In the loading course, sliding along the interfaces occurred. The friction along the contacted interfaces took part in resisting lateral load. If this horizontal crack was tension, no aggregate interlock effect could be expected. The reason is this crack is flexural crack not a shear crack; its width reached about 20mm at outmost end of the wall if in tension.

As shown in Fig. 5.1.1-1 to Fig. 5.1.1-6, though all the vertical reinforcement near the interface of the wall and foundation yielded, dowel action was not observed because no relatively large sliding along the horizontal crack occurred.

The diagonal reinforcement in tension yield at peak load as shown in Fig.5.1.1-23 to Fig.5.1.1-26. So the contribution of diagonal reinforcement in tension to the shear strength of the wall can be easily evaluated.

So it can be concluded that the shear force at the interface of the wall and foundation is mainly resisted by concrete in compression and diagonal reinforcement.

On the left lower corner a diagonal crack formed when lateral load reached $0.75 P_i$ as shown in Fig.5.1.2-1. In following loading cycles, this crack extended into base beam and width increased gradually. With increasing displacement levels

as shown in Fig.5.1.2-2, the diagonal cracks coming from upper corner on opposite end passed through this crack. This means that the diagonal concrete strut was serving as the first force resistant mechanism. However, the shear strength of the wall did not decrease when this zone was in compression. And the special thing is that these diagonal cracks coming from the other end intersected this diagonal crack with an approximate 90 deg angle. When the diagonal concrete strut was in compression, this diagonal crack almost closed completely, therefore, the strength of the wall was not damaged. When concrete crushing occurred in the lower left corners in following load cycles as shown in Fig.5.1.2-3, the strength of the wall did not decline. Finally, with fracture of the outermost vertical rebars at the lower right corner, the negative load value dropped. This phenomenon indicated that crushing of the concrete cover in a compression zone was not a critical factor in determining the strength of the wall if the concrete core in confinement is not damaged seriously.

Fig.5.1.2-1 Diagonal Crack in Left Corner at $0.75 P_t$

Fig.5.1.2-2 Diagonal Crack in Left Corner at DF=3

Fig.5.1.2-3 Diagonal Crack in Left Corner at DF=6

5.1.3 Crushing of Concrete and Buckling of Vertical Steel

When DF reached 6, fracture of vertical reinforcement at the lower right corner of W1 occurred as shown in Fig.5.1.3-1, and spalling of concrete cover can be seen from side view is serious, which means the depth of compression zone was decreased.

In course of testing W1, fracture of vertical bar occurred at the middle point between horizontal link and base beam and not at the base line level where, in theoretical calculation, the highest tension stress happened. This is due to the buckling of the vertical bar. There are three causes for the buckling of vertical bar.

- a. When load reversed, the opened flexural cracks closed gradually and the vertical bars previously yielded in tension under compression.
- b. At same time the lower right corner was under compression which caused horizontal dilation at this location. The dilation effect pushed the vertical bar out from the concrete. And the buckling of the vertical bar occurred in the plane of wall, no evident out-of-plane buckling was observed.
- c. Not enough horizontal restrictions were provided at this place. At the upper link and base beam, enough restraints were supplied; therefore, buckling did not occur at these positions.

The strain gauge was pasted near the fracture point; the record of this strain gauge can describe the strain history the vertical bar at this point as SVL10 shown in Fig. 5.1.1-6.

Concrete cover contacting channel at right lower corner started to spall when DF equal to 6 and the channel could be seen from S-side, which reduced the area of concrete under compression. But the strength of the specimen in positive direction did not decrease. Spalling of the concrete cover contacting channel was caused by pushing due to buckling of flange of channel. And the channel started to buckle at

right corner. At first cycle of higher displacement level, DF equal to 7, concrete cover fell off and a decrease of strength in positive direction could be seen from hysteresis loops of W1. In second cycle, breaking of weld of horizontal reinforcement took place, which means that the horizontal restraint was removed. As a result, the specimen experienced a pronounced decrease in strength.

During positive loading, concrete in the right lower corner was at a state of multi-axial compression. Horizontal reinforcement provided confining effect on the channel, when the horizontal bar broke, this effect was removed. As a result, compression acted on the unconfined concrete which was already cracked during previously loading course. And concrete was crushed as shown in Fig.5.1.3-2 with prominent drop in strength.

Fig.5.1.3-1 Fracture of Vertical Bar

Fig. 5.1.3-2 Buckling of Channel in Specimen W1

5.1.4 Strut-and-Tie Model

The aspect ratio of specimen W1 including top beam is 1.41 and the aspect ratio was 1.27 if only the wall panel was considered. According to the ACI 318-02 11.10, this kind of wall is a squat wall which frequently exhibits non-ductile behavior with sudden loss of strength and stiffness.

A.First Stage (DF less than 2)

Strain history of diagonal bars, as shown in Fig.5.1.23 to Fig.5.1.26, indicates that before ductility reached 2, the tension strain of diagonal bars was low. It means that the forces in diagonal bars were low. And the horizontal crack crossing the wall along the base line had not appeared at that stage. It means the contribution of the diagonal bars to resist lateral load was negligible. So the diagonal bars were not

taken into account in the following strut-and-tie models describing the load transfer in this stage.

In this specimen twenty vertical bars were almost evenly distributed along the length of the wall, and four channels were placed at boundaries of the specimen with two channels at each end. This layout of reinforcement indicated that main vertical steels are concentrated at ends of the specimen. According to the crack patterns shown in Fig. 4.2.4-1, the horizontal cracks, as hints of flexural deformations, were limited only in about 300mm from extreme fibers.

After all vertical bars in tension yield, assuming the depth of compression zone was 200mm at peak load, 16 vertical bars in tension yield, and bending moment due to these bars is $8 \times 2 \times 385 \times 78.5 \times 990 = 479 \text{ kNm}$.

If the vertical bars are simply divided into two parts with equal sectional area, 5×2 bars are in tension which located in boundaries of the wall, bending moment due to these bars is $5 \times 2 \times 385 \times 78.5 \times 1600 = 484 \text{ kNm} \approx 479 \text{ kNm}$

It hints that the two kinds of the distributions of vertical bars provide wall almost equal flexural capacity.

a. Simple Strut-and-Tie Model

From previous calculation, it is found that the flexural capacity of the wall with concentrated bars in boundaries is almost same to the wall with actual evenly distributed bars. A simple strut-and-tie model is developed as shown in Fig.5.1.4-1, in which the vertical bars are equally divided into two parts acting as concentrated bars placed in the boundaries of the wall. These vertical bars work together with channels located in same positions to resist lateral load.

With the assumed 200mm depth of compression zone when the all the vertical reinforcement in tension yield, the horizontal load is:

$$(856 \times 2 \times 310 + 5 \times 2 \times 78.5 \times 385) \times 1600 / 2540 = 525kN$$

In order to successfully transfer the lateral load from top beam to the base beam, enough amount of horizontal reinforcement is needed.

$\frac{525 \times 1000}{484.5 \times 78.5 \times 2} \approx 7$ and $\frac{2300}{7} = 328mm$ so the maximum spacing of horizontal reinforcement is 320mm.

Vertical reaction force at compression zone is:

$$C = \frac{525 \times 2540}{1600} = 833kN$$

Herein, the contribution of steel bars and channels in compression is ignored. Therefore all the reaction force is resisted by concrete in the compression zone.

$$\text{So the depth of compression zone is } d' = \frac{C}{0.85 f'_c b} = \frac{833 \times 1000}{0.85 \times 35.5 \times 140} = 197mm$$

C : vertical reaction force in compression zone,

f'_c : compressive strength of concrete cylinder;

b : thickness of wall;

d' : depth of compression zone;

- First mechanism

It is assumed that the diagonal concrete strut DB goes through the entire compression zone as shown in Fig.5.1.4-4.

The width of diagonal concrete strut:

$$w_s = d' \sin \theta = 197 \times \sin 35.8^\circ = 115.2mm$$

w_s : width of diagonal concrete strut;

θ : angle between diagonal concrete strut and horizontal.

From Fig.5.1.4-1, it can be seen that the shear capacity of specimen W1 is determined by the diagonal concrete strut with 35.8 deg to horizontal.

$$f_{sc} = \frac{C_{sc}}{w_s b} = \frac{525 \times 1.23}{115.2 \times 140} = 40 \text{ MPa} = 1.13 f'_c$$

f_{sc} :compression stress in diagonal concrete strut;

C_{sc} :compression in diagonal concrete strut;

This stress in the diagonal strut DB shows that this mechanism is incapable to resist the total lateral load by itself.

Maximum force in strut DB,

$$C_{DB} = 0.75 f'_c b w_s = 0.75 \times 35.5 \times 140 \times 115.2 = 429.4 \text{ kN}$$

Lateral load this mechanism can resist is: $P = C_{DB} / 1.23 = 429.4 / 1.23 = 349 \text{ kN}$

- Second mechanism

From the cracking patterns, an apparent diagonal concrete strut connecting load point and opposite lower corner formed between diagonal cracks as shown in Fig.5.1.4-2. The diagonal strut is a supplement to the first mechanism to resist lateral force.

Width of the diagonal concrete strut: $w_s = d' \sin \theta = 197 \times \sin 57.8^\circ = 166.7 \text{ mm}$

$$\text{Compression stress: } f_{sc} = \frac{C_{sc}}{w_s b} = \frac{525 \times 1.88}{166.7 \times 140} = 42.3 \text{ MPa} = 1.19 f'_c$$

The stress in the diagonal strut shows that this mechanism is unable to resist total lateral load by itself.

The second mechanism: maximum force in strut CB:

$$C_{CB} = 0.75 f'_c b w_s = 0.75 \times 35.5 \times 140 \times 166.7 = 621.4 \text{ kN}$$

Lateral load resisted by this mechanism is:

$$P = C_{CB} / 1.88 = 621.4 / 1.88 = 330.5 \text{ kN}$$

Total shear capacity: $349 + 330.5 = 679.5 \text{ kN}$

Safety factor $679.5 / 525 = 1.29$

The two mechanisms work together to resist the lateral load in the early loading

stage. With increasing displacement levels, overstrength of the vertical steel should be considered. The maximum lateral load obtained from experiment was 601kN, the safety factor is 1.13. In this calculation, the contribution of vertical steel in compression is not considered to resist vertical compression. If this contribution is taken into account, the depth of the compression zone will be less. Under cyclic load, especially under strong earthquake, the concrete cover in compression can to be damaged, which leads to decrease the depth of compression zone. This indicates that strong earthquake will cause a sudden failure with sudden loss of strength and stiffness and the wall will experience a non-ductile behavior when the maximum flexural capacity of the wall is reached.

Strut-and-tie models shown in Fig.5.1.4-1 and Fig.5.1.4-2 work simultaneously. A combined strut-and-tie model is developed incorporating the first and second mechanism according to the minimum energy dissipation theorem. The minimum energy dissipation theorem is principle of minimum strain energy for linear elastic behavior of the struts and ties after cracking as shown in following:

$$\sum F_i l_i \varepsilon_{mi} = \text{minimum}; \quad (5-1)$$

Where F_i = force in strut of tie i

l_i = length of member i

ε_{mi} = mean strain of member i

The contribution of the concrete struts can generally be omitted because the strains of the struts are usually much smaller than those of the steel ties.

Under same lateral load, strain energy dissipated by ties in the models in Fig.5.1.4.1 and Fig.5.1.4-2 is list in Table.5.1.4-1.

Table 5.1.4-1 Strain Energy in Strut-and-Tie of Specimen W1

Mechanism	Member	Force	Length	ε	Strain Energy	Contribution Rate
First	CD	87	1390	$0.55 \varepsilon_y$	$66.5 \varepsilon_y$	0.503
	DA	159	1150	ε_y	$182.85 \varepsilon_y$	
	DE	100	1600	ε_y	$160 \varepsilon_y$	
	$\sum F_i l_i$				$409.4 \varepsilon_y$	
Second	CA	159	2540	ε_y	$403.86 \varepsilon_y$	0.497

$\alpha_1 = E_2 / (E_2 + E_1) = 403.86 / (403.86 + 409.4) = 0.503$

$\alpha_2 = E_1 / (E_2 + E_1) = 409.4 / (409.4 + 403.86) = 0.497$

First mechanism: $525 \times 0.503 = 264kN < 349kN$

Second mechanism: $525 \times 0.497 = 261kN < 330.5kN$

Two strut-and-tie models shown in Fig.5.1.4-1 and Fig.5.1.4.2 take effect together. In order to obtain a mechanism with minimum strain energy, a combined strut-and-tie model as shown in Fig.5.1.4-3 is built based on models in Fig.5.1.4.1 and Fig.5.1.4-2 with appropriate proportion, in which 50.3 percent of the lateral force is resisted by the model shown in Fig.5.1.4-1, and the rest, 49.7 percent of the load, is taken by the model shown in Fig.5.1.4-2.

Fig.5.1.4-1 Simple Strut-and-Tie Model of Specimen W1

Fig.5.1.4-2 Additional Strut-and-Tie Model of Specimen W1

Fig.5.1.4-3 Combined Strut-and-Tie Model of Specimen W1

Fig.5.1.4-4 compression zone (by Yanez 1993)

b. Strut-and-Tie Model II

In the strut-and-tie model II, the contributions of vertical bars and channels to the flexural capacity are divided and calculated separately. In this situation, sixteen vertical bars and two channels yield in tension. This is useful to investigate the effect of the layout of vertical reinforcement on the ductility of the wall.

The distance between center of the bars in tension to the center of assumed compression was 990mm. Therefore, shear force caused by vertical bars:

$$(8 \times 2 \times 78.5 \times 385) \times 990 / 2540 = 188.5kN$$

And Shear force aroused by channels:

$$2 \times 856 \times 310 \times 1600 / 2540 = 334.3kN$$

The total shear force: $334.3 + 188.5 = 523kN \approx 525kN$

Reaction force in compression zone:

$$C = 8 \times 2 \times 78.5 \times 38.5 + 856 \times 2 \times 310 = 1014.3kN$$

Depth of compression zone:

$$d' = \frac{C}{0.85 f_c' b} = \frac{1014.3 \times 1000}{0.85 \times 35.5 \times 140} = 240mm$$

It can be seen that the flexural capacity of the wall when contribution of vertical bars and channels to flexural strength is calculated separately is almost equal to that of strut-and-tie model I in which the half vertical bars and channel are seen as concentrated bars placed in boundaries. However, the depth of compression zone is 240mm is twenty percent higher than that of strut-and-tie model I 197mm.

Similar procedures as shown in developing strut-and-tie model I in previous section are applied to develop the strut-and-tie models, which are built in accordance with vertical bars and channel, respectively. The strut-and-tie models only considering the vertical bars are shown in Fig.5.1.4-5 and Fig.5.1.4-6. The

strut-and-tie models only pertaining to channels are given in Fig.5.1.4-7 and Fig.5.1.4-8. Basing on the minimum energy dissipation theorem, the combined strut-and-tie models are developed similar to that in strut-and-tie model I as shown in Fig.5.1.4-7 and Fig.5.1.4-10. And the lateral loads act on the two combined strut-and-tie models according to the percentage of their flexural strength to the total flexural capacity of the wall, as shown in Fig.5.1.4-10 and Fig.5.1.4-11.

Table 5.1.4-2 Strain Energy in strut-and-tie of specimen W1

Mechanism	Member	Force	Length	ε	Strain Energy	Contribution Rate
First	EC	140	1390	$0.55 \varepsilon_y$	$107 \varepsilon_y$	0.57
	CA	256	1150	ε_y	$294.4 \varepsilon_y$	
	CD	100	990	ε_y	$99 \varepsilon_y$	
	$\sum F_i l_i$				$500.4 \varepsilon_y$	
Second	EA	256	2540	ε_y	$650.2 \varepsilon_y$	0.43

$\alpha_1 = 650.2 / (650.2 + 500.4) = 0.57$

$\alpha_2 = 500.4 / (650.2 + 500.4) = 0.43$

Fig.5.1.4-5 Strut-and-Tie Model I of Specimen W1 for Vertical bars

Fig.5.1.4-6 Strut-and-Tie Model II of Specimen W1 for Vertical bars

Fig.5.1.4-9 Strut-and-Tie Model II of Specimen W1

Fig.5.1.4-10 Combined Strut-and-Tie Model of Specimen W1

Fig.5.1.4-11 Strut-and-Tie Model of Specimen W1

Fig.5.1.4-12 Strut-and-Tie Model of Specimen W1

c. Strut-and-Tie Model III

In strut-and-tie model II the contributions of vertical bars and channels to total flexural capacity are calculated separately. And two combined models according to the flexural strength caused by vertical bars and channels are also drawn out in Fig.5.1.4-11 and Fig.5.1.4-12. However, this procedure is too tedious. Herein, the strut-and-tie model III considers the respective contributions of vertical bars and channels to the flexural capacity of the wall and uses a single vertical tie to replace vertical bars and channels at the center of the tension zone. The force in the tie is equal to sum of the vertical force caused by the vertical bars and that caused by the channels. The flexural capacity caused the tie is same as that caused by that of the vertical bars and channels. The assumed position of the vertical tie is at the centroid of the tension zone.

The vertical force caused by vertical bars:

$$8 \times 2 \times 78.5 \times 385 = 483.6 \text{ kN}$$

The vertical force caused by channels:

$$856 \times 2 \times 310 = 530.7 \text{ kN}$$

The total flexural capacity:

$$483.6 \times 0.99 + 530.7 \times 1600 = 1327.9 \text{ kN} \cdot \text{m}$$

The distance from center of tension zone to the center of compression zone (assuming depth of compression is 200mm same as that in previous sections):

$$1327.9 / (483.6 + 530.7) = 1.31 \text{ m}$$

Because the vertical tie in this strut-and-tie model is a substitute of vertical bars and channels. Therefore the strut-and-tie model III will have same flexural capacity and depth of compression as strut-and-tie model II. The strut-and-tie model III is simpler compared with strut-and-tie model II and has the identical shear reinforcement.

Table 5.1.4-3 Strain Energy in strut-and-tie of specimen W1

Mechanism	Member	Force	Length	ε	Strain Energy	Contribution Rate
First	CD	106	1390	$0.55 \varepsilon_y$	$81 \varepsilon_y$	0.53
	DA	194	1150	ε_y	$223.1 \varepsilon_y$	
	DE	100	1310	ε_y	$131 \varepsilon_y$	
	$\sum F_i l_i$				$435.1 \varepsilon_y$	
Second	CA	194	2540	ε_y	$492.8 \varepsilon_y$	0.47

$\alpha_1 = 492.8 / (435.1 + 492.8) = 0.53$

$\alpha_2 = 435.1 / (435.1 + 492.8) = 0.47$

Fig.5.1.4-13 Strut-and-Tie Model of Specimen W1

Fig.5.1.4-14 Strut-and-Tie Model of Specimen W1

Fig.5.1.4-15 Strut-and-Tie Model of Specimen W1

Fig.5.1.4-16 Combined Strut-and-Tie Model of Specimen W1

The width of the diagonal concrete strut is:

$$w_s = d' \sin \theta = 240 \times \sin 41.3^\circ = 158.4 \text{ mm}$$

$$C_{DB} = 0.75 f'_c b w_s = 0.75 \times 35.5 \times 140 \times 158.4 = 590.4 \text{ kN}$$

The shear force can be resisted by the first mechanism is:

$$590.4 / 1.33 = 443.9 \text{ kN}$$

$$w_s = d' \sin \alpha = 240 \times \sin 62.7^\circ = 213.3 \text{ mm}$$

$$C_{DB} = 0.75 f'_c b w_s = 0.75 \times 35.5 \times 140 \times 213.3 = 795 \text{ kN}$$

$$795 / 2.18 = 364.7 \text{ kN}$$

Shear capacity of this model:

$$P = 443.9 + 364.7 = 808.6 \text{ kN}$$

Safety factor:

$$808.6 / 525 = 1.54$$

The safety factor of the strut-and-tie model III is higher than that of than

strut-and-tie model I in which all the vertical reinforcement is placed in boundaries of the wall which has a similar flexural capacity to that of strut-and-tie model III. It indicates that in squat walls the evenly distributed reinforcement is helpful to improve ductility of the wall. If the concentrated reinforcement is placed in boundaries of squat wall, the wall will fail under cyclic loads after yielding of the flexural reinforcement with low ductility. Under a strong earthquake, the walls with concentrated reinforcement in boundaries will fail suddenly with non-ductile behavior due to the relative low depth of compression zone.

A. Second Stage (DF more than 4)

After ductility reached 4, the diagonal bars in the lower part of W1 yielded when they were in tension. And in following cycles, a wide horizontal crack passed through the interface between the wall and foundation. It indicated that these diagonal bars involved into resisting lateral load acting at top beam. This is discussed in later sections. In the strut-and-tie model describing performance in second stage, the contribution of diagonal bars to resist lateral load should be taken into account.

Due to the layout of the vertical reinforcement and channels and low aspect ratio, if no diagonal reinforcement is provided in this research, the specimen will experience sliding shear failure under cyclic reversed load. Under load reversals, the intersecting flexural cracks propagate across entire length of the wall at the base of the wall. As subsequent reversed loads are applied, the concrete near the base is destroyed by the breaking, abrasion and spalling. With the concrete destroyed, shear transfer by “truss action” is no possible and the horizontal reinforcement becomes ineffective. Interface shear transfer is lost. Shear sliding prevents the ductile behavior after yielding of the main vertical reinforcement. Deterioration of concrete

in this region is intensified by the vertical elongation of the wall, caused by the residual strain in the vertical reinforcement. These strains developed into the inelastic range with successive load reversals.

As observed experimentally by many researchers, in this case the applied shear is resisted mainly by friction and aggregate interlock between the cracked surfaces along the contact zone. For large deformations, dowel action of the longitudinal reinforcement also takes place. The sliding shear failure is not significantly affected by the level of confinement provided in ductile design. Diagonal reinforcement is perhaps the only successful solution for reducing the potential for sliding shear and enhancing the hysteretic characteristics of squat walls.

Other methods are investigated by other researchers, Saatcioglu and Murat (1990) experimentally, and by Wood and Sittipunt through FEM (1995). Herein only contribution of diagonal reinforcement to shear strength is studied.

The contribution of diagonal reinforcement in compression and in tension to the lateral load is taken into account by a diagonal strut and diagonal tie with a 45 deg angle to horizontal, respectively. The diagonal bars intersected at the mid-point of the base line. As shown in Fig.5.1.4-17, the acting lateral load resisted by the diagonal reinforcement was applied at the top beam. Considering the length of the diagonal reinforcement that extended into the wall from the base beam and the angle between the diagonal reinforcement and the horizontal, the diagonal strut and tie were placed in the strut-and-tie model. It is evident in Fig.5.1.4-17 the contribution of the diagonal reinforcement to resist lateral load. And it is shown in this figure that the diagonal reinforcement did not participate in resisting moment at the base level caused by lateral load and only contributed to the resistance of lateral load, which is the purpose of using diagonal reinforcement. This also verified that

vertical reinforcement located in the central part of the squat shear wall (wall web) was the primary lateral load resisting component in the late load cycles, especially when the vertical reinforcement in the boundaries of the wall experienced significant yielding.

The rest of the lateral load is resisted by the strut-and-tie model is shown in Fig.5.1.4-18. With increasing displacement levels, the elongation of the vertical steel, vertical rebars and channels, increased when they were in tension with wide flexural cracks near the base. When the load reversed, these wide flexural cracks that had opened previously, tended to close. When the lateral load reached zero, large residual deformations of the vertical reinforcement remained and the flexural cracks were still open. With increasing load, before the flexural cracks closed, the vertical force in the compression zone caused by the lateral load was resisted only this vertical reinforcement. When a limit of vertical load acting on this vertical reinforcement reached, these vertical steels buckled, which caused serious spalling of the cover concrete. Therefore, the compression zone became seriously damaged, which caused pronounced decrease in shear strength and stiffness of the wall. In the following cycles, the concrete confined in channels was compressed to failure when the horizontal reinforcement near the base link two channels together failed.

And with increasing displacement levels, the overstrength of the vertical steel occurred.

$$4 \times 78.5 \times 484.5 = 152.1 \text{ kN}$$

The lateral force resisted by diagonal reinforcement (the contribution of the diagonal reinforcement in compression is not considered):

$$152.1 \times \cos 45^\circ = 107.6 \text{ kN}$$

$$(107.6 \times 2) / 601 = 35.8\%$$

Shear force resisted by horizontal reinforcement:

$$525 - 107.6 \times 2 = 309.8 \text{ kN}$$

Number of horizontal bars:

$$309.8 \times 1000 / (2 \times 78.5 \times 484.5) = 4.1$$

$$215.2 \times 1000 / (2 \times 78.5 \times 484.5) = 2.8$$

$$107.6 \times 1000 / (2 \times 78.5 \times 484.5) = 1.4$$

Fig.5.1.4-17 Strut-and-Tie Model of Specimen W1

Fig.5.1.4-18 Strut-and-Tie Model of Specimen W1

5.1.5 Summary

A. The benefits of the evenly distributed vertical reinforcement:

1. When the wall with evenly distributed vertical reinforcement had the same flexural capacity as that of the wall with concentrated vertical reinforcement at the ends, the wall with evenly distributed vertical reinforcement exhibits more ductile behavior before concrete crushing failure occurs. The main root for this phenomenon is that depth of compression in wall with evenly distributed reinforcement when the wall reaches full flexural strength than that in wall with concentrate vertical reinforcement at boundaries, even though they have same amount of vertical reinforcement.
2. For walls with same amount vertical reinforcement and with same flexural capacity, when the walls reach their flexural capacity, more vertical reinforcement yields in tension in walls with evenly distributed vertical reinforcement than that in wall with concentrated vertical

reinforcement in boundaries of the wall, which causes different compression depths, influencing the width of diagonal concrete strut which is the main element to transfer lateral load to the foundation.

3. Due to the higher depth of compression in the wall with evenly distributed vertical reinforcement, there are higher safe residual than that in wall with concentrated reinforcement in boundaries.
4. From this analysis, it is better to place more vertical reinforcement in the central part of the wall if flexural capacity is satisfied, which can lead to deeper compression zone, finally produce wider diagonal concrete strut which means higher safe overplus for concrete crushing and make the wall exhibit more ductile behavior under cyclic load. This is useful in unpredictable earthquake.
5. Through analysis of strut-and-tie models, it is found that the shear force is mainly resisted by compression zone.

B. Benefits of diagonal reinforcement:

1. compared with the wall without diagonal reinforcement near the base, walls with diagonal reinforcement exhibit higher ductile behavior.
2. the contribution of diagonal reinforcement to resist lateral load took effect after the ductility DF, especially contribute more after appearance of the horizontal crack crossing the wall along the baseline in late cycles.
3. the diagonal reinforcement decrease the force in horizontal reinforcement, as a result, the force in diagonal concrete strut was decreased, which means that the concrete crushing failure of the concrete in the compression zone was postponed.

C. Benefits of confinement:

1. increase ductility of wall, because effect of confinement on improving

ductility is more pronounced than on improving strength.

- 2. confining effects increase strength of the compression zone after spalling of concrete cover. Concrete in confinement is not easy to be damaged and postpones compression failure of the compression zone, as a result, ductility is increased.

5.2 Behavior of Specimen W2

5.2.1 Strain History of the Gauges

A. Strain History of Vertical Bars

The strain histories of the vertical bars, SV3, SV5 and NV9, is shown in Fig.5.2.1-1 to Fig.5.2.1-3. These figures show that the records of strain gauges located at low level and middle level yielded when ductility reached 2. However, the deformation of the vertical rebars near the boundaries of wall was not so large to break these bars whereas it occurred in specimen W1 after serious yielding.

Fig. 5.2.1-1 Strain History of Vertical Bar SV3

Fig. 5.2.1-2 Strain History of Vertical Bar SV5

Fig. 5.2.1-3 Strain History of Vertical Bar NV9

B. Strain Profile of Vertical Bar at Base Line

The strain profiles of vertical bars near the base under negative and positive

load are shown in Fig. 5.2.1-4 and Fig.5.2.1-5, respectively. It can be seen from the two figures that in the first loading cycle, the assumption of plane section can be used. With increasing displacement levels, the distance from left side of the wall to the intersection between profiles and the base decreased accompanied with appearance of flexural cracks, and finally reaches a constant.

Fig.5.2.1-4 Strain Profile of Vertical Bars near the Base for Negative Load

Fig.5.2.1-5 Strain Profile of Vertical Bars near the Base for Positive Load

C. Strain History of Diagonal Bars

Strain histories of the diagonal bars near the base and in the horizontal connection region are shown in Fig.5.2.1-6 to Fig.5.2.1-13. Before reaching ductility of 2 (scan number 137), the strain of diagonal reinforcement was less than $2200 \mu\epsilon$, which means during this stage the contribution of these diagonal bars near the base is negligible. When the ductility reached 3 (scan number 230), the diagonal rebars near the base yield. It can be also seen from Fig. from ductility equal to 2, the diagonal reinforcement in the horizontal connection yields. The role of these rebars in resisting lateral load should be taken into account.

Fig.5.2.1-6 Hysteresis Loops of SD1

Fig.5.2.1-7 Strain history of SD1 versus Scan Number

Fig.5.2.1-8 Hysteresis Loops of ND3

Fig.5.2.1-9 Strain history of ND3 versus Scan Number

Fig.5.2.1-10 Hysteresis Loops of SUD5

Fig.5.2.1-11 Strain history of SDU5 versus Scan Number

Fig.5.2.1-12 Hysteresis Loops of NDL7

Fig.5.2.1-13 Strain history of NDL7 versus Scan Num

D. Strain History of Horizontal Reinforcement

Strain histories of the horizontal reinforcement are shown in Fig.5.2.1-14 to Fig.5.2.1-19. From these figures, it can be seen that almost all of the horizontal reinforcement yielded, which is different from the records of strain gauges of horizontal reinforcement in specimen W1, in which the horizontal reinforcement in the upper part of the wall did not experience yielding.

Fig.5.2.1-14 Strain History of Horizontal Bar NH11-2

Fig.5.2.1-15 Strain History of Horizontal Bar NH10-2

Fig.5.2.1-16 Strain History of Horizontal Bar SH9-2

Fig.5.2.1-17 Strain History of Horizontal Bar NH18-3

Fig.5.2.1-18 Strain History of Horizontal Bar NH4-2

Fig.5.2.1-19 Strain History of Horizontal Bar NH2-2

E. Strain History of Channel

Fig.5.2.1-20 Strain History of Horizontal Bar SS2-1

Fig.5.2.1-21 Strain History of Horizontal Bar SS2-2

Fig.5.2.1-22 Strain History of Horizontal Bar NS2-4

Fig.5.2.1-23 Strain History of Horizontal Bar NS2-5

Fig.5.2.1-24 Strain History of Horizontal Bar NS1-6

5.2.2 Load Transfer

In the early loading stages, the cracking patterns were similar to those of specimen W1 in same stage. It indicates that the internal forces of specimen W2 is similar to those in specimen W1 before ductility reached 2.

With increasing displacement levels, the cracks patterns were are different from those of specimen W1. The cracking patterns of specimen W2 in these stages exhibited two distinct features compared to those of specimen W1. First, diagonal cracks formed in the upper and lower wall panels of W2, which made the wall seem as two independent wall panels. Second, the range of diagonal cracks at the top beam level spread over half the width of the wall whereas these kinds of cracks were limited in a smaller range in specimen W1. This means that a diagonal concrete strut DE started approximately from center of the top beam, as shown in Fig.5.2.4-2, and the tie DC to balance this concrete strut was part of the vertical reinforcement in the upper wall panel. This means that the existence of the

horizontal connection changed the aspect ratio of the wall, especially in upper wall panel, which finally changed the internal force paths in whole specimen.

Readings of the strain gauges placed on the diagonal bars in the horizontal connection showed that these diagonal bars yielded after a global ductility of 2. However, after the appearance of a horizontal crack along the horizontal connection, no additional deformation or damage of the horizontal connection was observed, which indicated that the horizontal connection successfully transferred the load from the upper wall panel to the lower one. In order to simplify the procedure of developing a strut-and-tie model, the connection zone was defined as the region in the upper or lower panel containing diagonal reinforcement, and the compression forces in the concrete struts were assumed to transfer through it. In another words, the connection zone was assumed to be continuum.

The wide horizontal crack crossing through the wall along the interface with the foundation, which occurred in specimen W1 was not observed in specimen W2. However, two diagonal cracks formed just above diagonal reinforcement near the base intersect at 125 deg angle. The protruding concrete tip and the horizontal bars crossing it formed the main shear resistant mechanism under cyclic load. In later loading cycles, the concrete in compression at the ends of the diagonal reinforcement near the base experienced spalling as shown in Fig.5.2.3-1, which caused a significant decrease in the compression area of the diagonal concrete struts at this location. As a result, it decreased the shear resisted by the concrete. Therefore, shear resistance requirement of the horizontal and vertical reinforcement increased, which finally caused fracture of the horizontal reinforcement. This indicates that in shear design for cyclic load, the contribution of concrete to shear should be ignored, especially in squat shear walls.

In early stages, strut DB shown in Fig.5.2.4-1, was the main shear resistant

member, whereas in the later loading stages, strut HI shown in Fig.5.2.4-4, was the main member to resist lateral load. There was a shift of critical location for resisting lateral load during the course of loading.

Records of the strain gauges located on diagonal reinforcement near the base showed that diagonal reinforcement near the base yielded. The contribution of these diagonal bars to the lateral load separated into forces in tie LK and concrete strut GL as shown in Fig.5.2.4-4.

5.2.3 Crushing of Concrete and Buckling of Vertical Steel

After reaching a ductility of 2, the critical compression zone shifted from the lower corners of the wall to the up ends of diagonal reinforcement near the base.

As shown in Fig.5.2.4-4, after ductility reached 2, a diagonal strut HI from loading point to end of diagonal reinforcement near the base formed. The density of the diagonal cracks at this deformation area was high. Under cyclic loading, concrete at the up end of the diagonal reinforcement near the base deteriorated and spalled. After several loading cycles, the total area of compression struts at this location decreased due to spalling. With increasing load, it can be observed that the diagonal concrete bands in compression at the height level 650mm from the wall foundation interface were sheared one by one, which caused increment in tensile forces horizontal reinforcement and finally fracture the horizontal reinforcement. And the specimen at last experienced shear failure.

When the horizontal reinforcement crossing the protruding concrete tip which resisted lateral force yielded, the channel embedded in the wall buckled at height level where horizontal reinforcement broke.

The different crushing location of W2 compared to that of the integral wall W1

was mainly caused by the existence of the horizontal connection, which changed the internal load paths through which lateral load acting on top panel could be transferred to the bottom panel.

Fig.5.2.3-1 Crushing of Concrete in Specimen W2

Fig.5.2.3-2 Crushing of Concrete in Specimen W2

5.2.4 Strut-and-Tie Model

Before ductility of the wall reached 2, a horizontal crack developed across the horizontal connection, however, no evidence of sliding was observed along the connection. The crack patterns were similar to those of specimen W1 at similar displacement levels. From records of strain gauges placed on the diagonal rebars near the base as shown in Fig.5.2.1-7 and Fig.5.2.1-9, the contribution of these diagonal bars to lateral load resistance was negligible before reaching the ductility of 2. Therefore, in this stage the strut-and-tie mode shown in Fig.5.2.4-1 is applicable.

Fig.5.2.4-1 Strut-and-Tie Model for W2

After reaching a ductility of 2, the diagonal cracks belonging to the upper and the lower panels were observed. It indicated changes of the internal load paths of W2. Due to change of aspect ratio of the upper wall panel, the upper panel was seemed to be monolithic wall and the connection zone was considered as its foundation. In this stage, the aspect ratio of the upper panel was 0.59 much lower than 1. Referring to the cracking patterns in this panel, a concrete strut DE was added. Due to the changes in force paths in the upper panel, the internal paths were different from those of specimen W1.

When the diagonal reinforcement near the base was taken into effect in resisting lateral load, the contribution of these diagonal rebars was shown in Fig.5.2.4-4. When the diagonal bars were in tension, they were represented by a diagonal tie. When they were in compression, the concrete took part in resisting the compression; therefore, a diagonal concrete strut was used.

In Fig.5.2.4-3, lateral load was transferred to the base through the concrete strut G’F. In Fig.5.2.4-4, in this mechanism, tie KI transferred the lateral load to tie GK and LK. The internal forces in tie GK and LK were proportional to their total sectional area.

Table 5.2.4-1 Strain Energy in strut-and-tie of specimen W2

Mechanism	Member	Force	Length	ε	Strain Energy	Contribution Rate
First	AG'	158.8	800	ε_y	$127 \varepsilon_y$	0.52
	CD	133.8	1290	ε_y	$172.6 \varepsilon_y$	
	BE	100	1600	ε_y	$160 \varepsilon_y$	
	BG'	86.9	650	$0.5 \varepsilon_y$	$28.2 \varepsilon_y$	
	G'I'	50	1600	$0.5 \varepsilon_y$	$40 \varepsilon_y$	
	$\sum F_i l_i$				$527.9 \varepsilon_y$	
Second	AG	181.4	650	ε_y	$117.9 \varepsilon_y$	0.48
	GN	118	850	$0.65 \varepsilon_y$	$65.2 \varepsilon_y$	
	NH	83.9	1040	$0.46 \varepsilon_y$	$40.1 \varepsilon_y$	
	GK	78	1450	$0.78 \varepsilon_y$	$88.2 \varepsilon_y$	
	LK	31	919	ε_y	$28.5 \varepsilon_y$	
	KI	100	150	ε_y	$15 \varepsilon_y$	
	NM	100	1250	ε_y	$125 \varepsilon_y$	
	$\sum F_i l_i$				$479.9 \varepsilon_y$	

Based on the minimum strain energy theory, the lateral load was distributed according to the two mechanisms.

$$E_2 / (E_2 + E_1) = 0.48$$

$$E_1 / (E_2 + E_1) = 0.52$$

The lateral force resisted by the first mechanism was:

$$0.48 \times 525 = 252 \text{ kN}$$

The second mechanism resists lateral load:

$$525 - 252 = 273 \text{ kN}$$

The contribution of the two mechanisms to resist lateral load was combined into one strut-and-tie model, as shown in Fig.5.2.4-7 in which the two mechanisms took effect simultaneously with appropriate proportion, which is drawn in Fig.5.2.4-5 and Fig.5.2.4-6. From the combined strut-and-tie model, all the concrete struts concentrated at point F formed a three-dimensional compression zone. And at point I a C-C-T node zone formed.

Based on the combined strut-and-tie model, the amount of horizontal reinforcement needed in upper and lower wall panels was:

For upper panel:

$$252 \text{ kN} / (78.5 \times 2 \times 484.5) = 3.1$$

$$\text{Safety factor: } 5 / 3.1 = 1.6$$

For lower panel:

$$273 \text{ kN} / (78.5 \times 2 \times 484.5) = 3.59$$

Safety factor:

$$5 / 3.59 = 1.39$$

For diagonal reinforcement near the base,

$$84.63 \text{ kN} / (78.5 \times 2 \times 484.5) = 1.1$$

Safety factor:

$$2 / 1.1 = 1.8$$

Comparing these safety factors for reinforcement placed in the specimen, it can

be seen that the tie KI was the critical component in the strut-and-tie model. Considering the overstrength of the vertical reinforcement, the capacity of the wall was 576.96kN.

$$576.96/525 = 1.1$$

This result shows that when the wall reached its capacity, the horizontal tie HI yielded and initiated plastic deformation, at the same time the contribution of the concrete to the lateral load decreased. These two factors caused the failure of the wall.

From the calculations above, the amount of horizontal reinforcement for the upper wall panel was less than that compared to specimen W1 and more horizontal reinforcement was demanded in the connection located in the lower panel.

Fig.5.2.4-2 Strut-and-Tie Model for Upper Wall Panel and Connection Zone in Upper Panel

Fig.5.2.4-3 First Strut-and-Tie Model of Specimen W2

Fig.5.2.4-4 Second Strut-and-Tie Model of Specimen W2

Fig.5.2.4-5 First Strut-and-Tie Model of Specimen W2 under Proportioned Load

Fig.5.2.4-6 Second Strut-and-Tie Model of Specimen W2 under Proportioned Load

Fig.5.2.4-7 Combined Strut-and-Tie Model of Specimen W2

To predict the behavior of the horizontal connection as shown in Fig.5.2.4-8(a), as studied in section 4.3.2 for design, the flexural deformation of the horizontal connection was due to tension and compression deformation, and shear deformation only comes from deformation of shear connector shown in Fig.5.2.4-8(b). If the tolerance between bolts and bolt holes is ignored, the behavior of the horizontal connection can be simulated by the model shown in Fig.5.2.4-8(c).where K1, K2 and K3 represent the stiffness of the tension, compression and shear springs in the model, respectively.

$$K1 = K2 = \frac{l}{A_e E_s}$$

Where l is the height of connection zone; A_e is the effective area of tension tie;
 E_s is elastic modulus of steel.

$$K3 = \frac{l}{A_d E_s \sin \alpha \cos \alpha}$$

Where A_d is total area of diagonal reinforcement in tension; α is the incline angle of diagonal tension bars.

When tolerance between bolts and bolt holes exist and are taken into account, a reduced value of stiffness should be used.

Fig.5.2.4-8 Behavior of Horizontal Connection(cont')

Fig.5.2.4-8 Behavior of Horizontal Connection

5.2.5 Summary:

- In the early stages of loading, when the ductility was less than 2, the crack patterns of specimen W2 were similar to those of specimen W1 at the same displacement levels. It means that in the early stage the precast wall behaved as an integral wall. It also indicates that the internal load paths were similar to those of specimen W1 at this stage.
- In later loading cycles, spalling of the concrete at the ends of the diagonal reinforcement near the base caused a decrease in the area of the compression zone at this location which led to a decrease in the contribution of the concrete to the lateral load. Simultaneously, the forces in the horizontal reinforcement increased and finally caused fracture of these bars at a height level that corresponded to the end of the diagonal reinforcement near the base.
- In strut-and-tie model developed for W1, the force in the horizontal tie could distributed in more horizontal reinforcement with respect to the number of the involved horizontal bars of the lower wall panels of W2. It was consistent with the developed strut-and-tie models of W2. It showed that more reinforcement were in need in between the diagonal reinforcement in the connection and that near the foundation to arrest the failure of specimen W2.

- When the ductility was larger than 2, the internal load paths were different from those in the specimen W1 at the same displacement level. This situation was due to the existence of the horizontal connection in the precast wall W2. In specimen W2, the aspect ratio of the upper wall panel was 0.6. In later loading cycles, the diagonal cracks in the upper panel spread to the center of the wall at top beam level, which meant that a diagonal concrete strut started to form from the center of the top beam.
- Over the whole course of loading, deformation or damage of the horizontal connection was not observed, which indicates that the lateral load acting on the top beam was successfully transferred from the upper wall panel to the lower one.
- Specimen W2 experienced a different failure mode and failure location than that from those of specimen W1.
- Contribution of concrete to lateral load resistance under cyclic load should not be taken into account, especially in a squat wall.
- Compared with specimen W1, less horizontal reinforcement is demanded in the upper part of specimen W2. And more horizontal reinforcement was needed in the connection zone located in the lower panel.
- Strong base wall is needed for the precast wall due to its high requirement on shear strength.

5.3 Behavior of Specimen W3

Specimen W3 was another precast counterpart of specimen W1. The only difference between specimen W2 and W3 was the shear connectors embedded in wall panels. The rest of the details including reinforcement, channels are same to

those in specimen W2.

5.3.1 Strain History of Gauges

A. Strain History of Vertical Bars

Records of the strain gauges placed on the vertical bars are shown in Fig.5.3.1-1 to Fig.5.3.1-3. It can be seen from these figures that all the readings noted by L were much bigger than 2700, which means that all the vertical bars yielded at the baseline level. Some the records label as M yielded during the loading course while the others did not, such as SVM6. It indicates that at the M level there was elastic core whereas the two sides of wall yielded. The vertical bars labeled as H experienced less deformation as shown in these figures, which is a result of cutting of vertical bars into two at the horizontal connection. All these features of the vertical bars are similar to those in specimen W2. It hints that the type of shear connectors made no much difference on the performance of vertical bars.

Fig. 5.3.1-1 Strain History of Vertical Bar SV1

Fig. 5.3.1-2 Strain History of Vertical Bar SV3

Fig. 5.3.1-3 Strain History of Vertical Bar NV6

B. Strain Profiles of Vertical Bar at Base Line

Profiles of vertical bars near the base are shown in Fig.5.3.1-4 to Fig.5.3.1-5 for

negative and positive loading. It can be seen that in the first cycle, namely under a load of $0.25 P_i$, the strain profile exhibited almost a linear trend. With increasing load this trend changed. It also shows that the depth of compression zone increased with the increasing load and became stable when the load reached yielding load at about 190mm from the extreme in compression fiber.

Fig.5.3.1-4 Strain Profile of Vertical Bars near the Base for Negative Load

Fig.5.3.1-5 Strain Profile of Vertical Bars near the Base for Positive Load

C. Strain History of Diagonal Bars

The strain history of diagonal bars is shown in Fig.5.3.1-6 and Fig.5.3.1-8. To some extent, these figures indicate the energy dissipation capacity of these bars. The relationships of diagonal bar strain versus scan number are shown in Fig.5.3.1-7 and Fig.5.3.1-9. It shows that diagonal bar ND1 yielded at scan number 181 and ND4 yielded at scan number 157. These bars showed negligible negative values.

The strain history of the diagonal bars welded on the steel shear connectors and relationships of strain versus scan number are shown in Fig.5.3.1-10 to Fig.5.3.1-17. These bars include the diagonal bars embedded in the upper and lower wall panels and the different loading directions. All of the diagonal bars placed in the connection zone yielded after the specimen went into plastic stages.

Fig.5.3.1-6 Strain History of Diagonal Bar ND1

Fig.5.3.1-7 Relationship Strain versus Scan Number of ND1

Fig.5.3.1-8 Strain History of Diagonal Bar ND4

Fig.5.3.1-9 Relationship Strain versus Scan Number of ND4

Fig.5.3.1-10 Strain History of Diagonal Bar NDU6

Fig.5.3.1-11 Relationship Strain versus Scan Number of NDU6

Fig.5.3.1-12 Strain History of Diagonal Bar SDU5

Fig.5.3.1-13 Relationship Strain versus Scan Number of SDU5

Fig.5.3.1-14 Strain History of Diagonal Bar NDL2

Fig.5.3.1-15 Relationship Strain versus Scan Number of NDL2

Fig.5.3.1-16 Strain History of Diagonal Bar SDU2

Fig.5.3.1-17 Relationship Strain versus Scan Number of SDU2

D. Strain History of Horizontal Bars

Relationships of strain versus scan number for the horizontal reinforcement are shown in Fig.5.3.1-18 to Fig.5.3.1-24. It can be seen from these figures that all the horizontal reinforcement in the upper wall panel did not yield during the loading course, whereas some horizontal reinforcement in the lower wall panel yielded. It can also be seen that the horizontal reinforcement near the base near the diagonal reinforcement crossing the wall-foundation interface experienced lower deformation in center of the wall compared to that of W1. One of the main factors affecting the deformations of these horizontal bars near the foundation was the diagonal bars which resisted part of the shear force; the other was due to the constraint of the foundation block. The deformation characteristics of the horizontal reinforcement placed in the upper and lower wall panels are represented in the strut-and-tie models shown in Fig.5.2.24-7.

Compared to the horizontal reinforcement in specimen W1, the shear deformation of the horizontal reinforcement in the lower wall panel was increased in the later loading stages while those in the upper wall panel decreased.

Fig.5.3.1-18 Relationship Strain versus Scan Number of SH1-2

Fig.5.3.1-19 Relationship Strain versus Scan Number of SH3-2

Fig.5.3.1-20 Relationship Strain versus Scan Number of NH5-2

Fig.5.3.1-21 Relationship Strain versus Scan Number of NH8-2

Fig.5.3.1-22 Relationship Strain versus Scan Number of SH9-2

Fig.5.3.1-23 Relationship Strain versus Scan Number of NH10-2

Fig.5.3.1-24 Relationship Strain versus Scan Number of SH12-2

E. Strain History of Channels

The strain history of the channels that acted as concentrated flexural reinforcement is shown in Fig.5.3.1-25 to Fig.5.3.1-28. Deformation mainly occurred in the lower part of the wall and exhibited a decreasing trend from the bottom to the top of the wall. There was no plastic deformation in the channels embedded in upper wall panel.

The hysteresis loops of NS were similar to those of the flexural bars at the same location. The configuration of the loops means that much more energy dissipated through flexural deformation of the vertical channel in lower wall panel than in upper wall panel.

Fig.5.3.1-25 Strain History of Channel NS1-1

Fig.5.3.1-26 Strain History of Channel NS1-4

Fig.5.3.1-27 Strain History of Channel NS1-5

Fig.5.3.1-28 Strain History of Channel NS1-6

5.3.2 Load Transfer

In the early loading stages before the ductility reached 2, though the horizontal crack along the horizontal connection appeared, this specimen still behaved as an integral wall, which can be seen from the cracking patterns.

After the ductility reached 2, the cracking pattern changed. They showed that wall panels behaved as independent squat walls with diagonal cracks crossing the wall panels from corner to the opposite diagonal corner.

In late loading cycles, shear deformation of the wall mainly occurred in the lower walls panel, and the density of the flexural and shear cracks increased. The crack density at the upper ends of the diagonal reinforcement near the base was much higher than any other location. Under cyclic loading, spalling of the concrete at this location occurred. The concrete spalling at this location decreased the area of

concrete strut in compression, which caused increased stress in horizontal reinforcement passing through region. At this time, these horizontal bars already yielded and plastic deformation increased fast. As a result, the reinforcement in the plastic stage could not prevent further shear deformation at this height level.

Fig.5.3.2-1 Fracture of Channels

5.3.3 Strut-and-Model

As mentioned before, the only difference between specimen W3 and W2 was the shear connectors. Through investigating the load transfer of specimen W3, it exhibited similar behavior to W2. In the early loading stages, specimen W2 behaved like an integral wall, the strut-and-model shown in Fig.5.2.4-1 was developed to indicate the flow of the internal forces. With increasing displacement levels, the upper and lower wall panels behaved independently, and the horizontal connection transferred the lateral load to the lower wall panel successfully before fracture occurred in channels. The failure of the horizontal reinforcement occurred near the

connection zone caused that the lateral load could not transfer completely by the shear connectors. As a result, the lateral load was transferred by the channel connections caused the fracture of the channels. The strut-and-tie models shown in Fig.5.2.4-5 and Fig.5.2.4-6 were developed for this stage.

5.4 Behavior of Specimen W4

Specimen W4 was an integral wall with embedded I-beams which were used as vertical flexural reinforcement in the boundaries of the wall. The difference between specimen W4 and W1 was just the different embedded steel shapes.

5.4.1 Strain History of Gauges

A. Strain History of Vertical Bars

The strain history of the vertical bars is shown in Fig.5.4.1-1 to Fig.5.4.1-5. It can be seen from these figures that all vertical bars yielded at the wall-foundation interface. Almost all of the vertical reinforcement noted by M yielded which was located 750mm from the interface.

Fig.5.4.1-1 Strain History of Vertical Bars SV1

Fig.5.4.1-2 Strain History of Vertical Bars NV2

Fig.5.4.1-3 Strain History of Vertical Bars SV6

Fig.5.4.1-4 Strain History of Vertical Bars NV7

Fig.5.4.1-5 Strain History of Vertical Bars NV9

B. The Strain Profiles of the Vertical Bars

Strain profiles of the vertical bars along the wall-foundation interface are shown in Fig.5.4.1-5 and Fig.5.4.1-7.

Fig.5.4.1-6 Strain Profiles of Vertical Bars for Negative Load

Fig.5.4.1-7 Strain Profiles of Vertical Bars for Positive Load

C. Relationship of Strain versus Scan Number of Diagonal Bars

Relationships of strain versus scan number of diagonal bars are shown in Fig.5.4.1-8 and Fig.5.4.1-9. Strains in diagonal bars early loading stage, that is, at ductilities less than 2, were lower than 2700, which means that contribution of these diagonal bars to lateral load was negligible. However, after ductility reached 2, the strain in diagonal reinforcement increased and yielded. This means that the deformation of the diagonal bars increased, especially, in later cycles after the appearance of a wide horizontal crack crossing through wall along the interface with the foundation.

Fig.5.4.1-8 Relationship of Strain versus Scan Number of SD1, ND1

Fig.5.4.1-9 Relationship of Strain versus Scan Number of SD3, ND3

D. Relationship of Strain versus Scan Number of Horizontal Bars

The strain history of the horizontal reinforcement is shown in Fig.5.4.1-10 to Fig.5.4.1-15. It can be seen that in range of the diagonal reinforcement, the strain

readings of the horizontal reinforcement were lower than 2700, which means that the lateral load resisted by diagonal reinforcement was low. As a result, in this region, the diagonal and the horizontal reinforcement worked together to resist the lateral load. But readings of NH9, NH8 and NH6 were more than 2700, which indicates these horizontal bars yielded during the loading course. Horizontal reinforcement in higher levels did not yield.

Fig.5.4.1-10 Relationship of Strain versus Scan Number of NH11

Fig.5.4.1-11 Relationship of Strain versus Scan Number of SH9, NH9

Fig.5.4.1-12 Relationship of Strain versus Scan Number of NH8

Fig.5.4.1-13 Relationship of Strain versus Scan Number of NH6

Fig.5.4.1-14 Relationship of Strain versus Scan Number of SH3

Fig.5.4.1-15 Relationship of Strain versus Scan Number of NH2

5.4.2 Load Transfer

In the early stages, the flexural cracks were limited in a range of 200mm from extreme fiber of the wall and several cracks crossed the wall from the bottom corner to opposite the top corner with a concrete band width about 200mm. It meant that in this stage the lateral load was resist mainly by the diagonal concrete strut.

After reaching a ductility of 1, cracks appeared along the inner boundaries of I-Beams in the lower parts of W4. With increasing load, the range of this kind of cracks extended upward and the density of the flexural cracks increased in the lower part of the wall. When ductility reached 3, the spalling of the concrete occurred along the inner boundaries of the I-Beams in the lower parts of the wall. With increasing displacement levels, concrete spalling spread upward and finally almost extended to the top beam as shown in Fig. 4.2.4-4. This phenomenon indicated that to some extent, the nodal area of the diagonal concrete bands in compression decreased.

A large horizontal crack occurred which passed through the wall at the interface with the foundation when ductility reached 5. And at this stage, the readings of the strain gauges placed on the diagonal bars indicated them already yielded. The density of the diagonal cracks in the wall increased and the horizontal reinforcement yielded.

5.4.3 Concrete Crushing and Buckling of I-Beam

When the lateral load reached P_i , a number of cracks appeared along the inner boundaries of the I-Beams in the lower part of W4. After reaching a ductility of 4, the cracks proliferated and spread from the bottom of the wall to its top. The cracks intersected the horizontal cracks and the diagonal cracks from the top corner of the wall. Under cyclic loading, spalling of the concrete occurred along the inner boundaries of the I-Beams. When ductility reached 5, a vertical crack appeared at the lower left corner of the wall, which caused spalling of the concrete cover as shown in Fig.5.4.3-1, to some extent, decreased the lever arm when left lower corner was in compression.

The spalling of the concrete along the inner boundaries of the I-Beam grew with the increasing loading cycles and influenced the force transfer between the I-Beams and the contacting concrete, herein, call it bond-failure as shown in Fig.5.4.3-2. The bond failure led to a result that the lateral load was transferred to the foundation mainly by the diagonal concrete struts. Due to crushing of concrete cover in the lower corner, the nodal areas of the concrete strut in compression decreased. Finally, concrete nodes in compression were crushed with buckling of the I-Beam in the lower corner.

Fig.5.4.3-1 Spalling of Concrete Cover in Compression Zone

Fig.5.4.3-2 Spalling of Concrete along the Inner Boundaries of I-Beams

5.4.4 Strut-and-Tie Model

In the early loading stages, the diagonal cracks started from the bottom corner to the opposite top corner of the wall. The strain records of the diagonal bars placed near the foundation showed that the stress in the diagonal bars was low which

meant the contribution of the diagonal bars to resist the lateral load was negligible. Based on readings of the strain gauges placed on the horizontal reinforcement, two mechanisms were assumed to simulate the behavior of specimen W4 in this stage. Fig.5.4.4-2 to Fig.5.4.4-4

In the later loading stages, a large horizontal crack crossed the wall along the interface with the foundation. The diagonal bars located near the foundation of the wall passing through the horizontal crack took part in resisting lateral load. Considering the records of the strain gauges on the diagonal bars, as discussed in previous section, the contribution of the diagonal reinforcement was taken into account in developing strut-and-tie model of specimen W4 in the later loading stage. Strut-and-tie model shown in Fig.5.4.4-5 considered the contribution of the diagonal bars, in which yield strength of the diagonal bars were used according to the records of strain gauges placed on them. Lateral load was proportionally distributed to the individual strut-and-tie model shown in Fig.5.4.4-5 and Fig.5.4.4-6.

The total vertical force:

$$T_1 = 322.16 \times 2050 = 660.4kN$$

$$T_2 = 8 \times 2 \times 78.5 \times 384.5 = 483kN$$

$$T = T_1 + T_2 = 1143.4kN$$

The compression depth:

$$d = \frac{T}{0.85 f_c b} = \frac{1143.4 \times 1000}{0.85 \times 33.53 \times 140} = 287mm$$

Distance of center of gravity of vertical bars in tension to the center of compression zone:

$$1800 - 710 - 144 = 946mm$$

$$w_s = d' \sin \alpha = 316 \times \sin 41.5^\circ = 209.4 \text{ mm}$$

$$C_{DB} = 0.75 f_c' b w_s = 0.75 \times 33.5 \times 140 \times 209.4 = 763.6 \text{ kN}$$

The shear force can be resisted by the first mechanism is:

$$763.6 / 1.336 = 551.3 \text{ kN}$$

$$w_s = d' \sin \alpha = 316 \times \sin 62.7^\circ = 280.8 \text{ mm}$$

$$C_{DB} = 0.75 f_c' b w_s = 0.75 \times 33.5 \times 140 \times 280.8 = 978 \text{ kN}$$

$$978 / 2.197 = 449.6 \text{ kN}$$

Shear capacity of this model:

$$P = 551.3 + 449.6 = 1001 \text{ kN}$$

Safety factor:

$$1001 / 584.4 = 1.7$$

Fig.5.4.4-1 Center of Gravity of Vertical Bars in Tension

The lateral load:

$$P = \{(1800 - 100 - 144) \times 660.4 + 946 \times 483\} / 2540 = 584.4 \text{ kN}$$

Distance of the centroid of all of the vertical steels to the mid-line of compress zone:

$$\frac{584.4 \times 2540}{(660.4 + 483)} = 1298 \text{ mm}$$

Table 5.4.4-1 Strain Energy in strut-and-tie of specimen W4

Mechanism	Member	Force	Length	ε	Strain Energy	Contribution Rate
First	CD	107	1390	$0.55 \varepsilon_y$	$81.8 \varepsilon_y$	0.53
	DA	195.6	1150	ε_y	$224.9 \varepsilon_y$	
	DE	100	1298	ε_y	$130 \varepsilon_y$	
	$\sum F_i l_i$				$436.7 \varepsilon_y$	
Second	CA	195.6	2540	ε_y	$496.8 \varepsilon_y$	0.47

Lateral load resisted by the first mechanism:

$P_1 = 0.55P = 0.55 \times 584.4 = 321.4kN$

Lateral load resisted by the second mechanism:

$P_2 = 0.47P = 0.47 \times 584.4 = 263kN$

Fig.5.4.4-2 Strut-and-Tie Model of W4

Fig.5.4.4-3 Strut-and-Tie Model of W4

Fig.5.4.4-4 Combined Strut-and-Tie Model of W4

Fig.5.4.4-5 Strut-and-Tie Model of W4

Fig.5.4.4-6 Strut-and-Tie Model of W4

5.5 Behavior of Specimen W5

5.5.1 Strain History of the Gauges

A. Strain History of Vertical Bars

The strain history of the vertical bars was shown in Fig.5.5.1-1 to Fig.5.5.1-5 by strain vs. scan number. It can be seen from the figures that all of the vertical bars noted by L and M yielded. However, the vertical bars in the upper panel did not yield in the early loading stages.

Fig.5.5.1-1 Relationship of strain Versus Scan Number of Vertical Bar SV1

Fig.5.5.1-2 Relationship of strain Versus Scan Number of Vertical Bar NV2

Fig.5.5.1-3 Relationship of strain Versus Scan Number of Vertical Bar SV3

Fig.5.5.1-4 Relationship of strain Versus Scan Number of Vertical Bar NV4

Fig.5.5.1-5 Relationship of strain Versus Scan Number of Vertical Bar SV5

B. Strain Profiles of Vertical Bars

The strain profiles of the vertical bars in first 5 cycles were shown in Fig.5.5.1-6 and Fig.5.5.1-7 for the positive and the negative load, respectively. In the

first cycle, for the positive and the negative load, the strain profiles exhibited linear trends. With increasing load, the depth of the compression zone increased and the compression strain at the compressive extreme fiber also increased. In the fifth cycle, the linear trends of the strain profiles were totally lost.

Fig.5.5.1-6 Strain Profiles of vertical Bar for Positive Load

Fig.5.5.1-7 Strain Profiles of vertical Bar for Negative Load

C. Strain History of Diagonal Bars

Hysteresis loops of the strain of the diagonal bars near the foundation vs. load were shown in Fig.5.5.1-8 and Fig.5.5.1-10. The diagonal bars near the foundation exhibited some strain energy dissipation capacity. Relationships of diagonal reinforcement near the base versus scan number were shown in Fig.5.5.1-9 and Fig.5.5.1-11. From Fig.5.5.1-9 and Fig.5.5.1-11 it can be seen that the diagonal bar SD1 yielded at the scan number 310 and ND4 yielded at the scan number 348, which meant that before reaching a ductility of 3, the diagonal reinforcement near the foundation did not yield and its contribution to lateral load resistance was negligible. After ductility reached 3, the diagonal bars near the foundation yielded.

The characteristics of the strain histories of the diagonal bars in the horizontal connection were shown in Fig.5.5.1-12 to Fig.5.5.1-19. Compared with diagonal bars near the foundation, the diagonal bars in the horizontal connection yielded earlier compared to the diagonal bars near the foundation. And from the features of the hysteresis loops of all of the diagonal bars, the diagonal bars in the horizontal connection exhibited much lower energy dissipation capacity.

Fig.5.5.1-8 Strain History of SD1

Fig.5.5.1-9 Relationship of Strain versus Scan Number of SD1

Fig.5.5.1-10 Strain History of ND4

Fig.5.5.1-11 Relationship of Strain versus Scan Number of ND4

Fig.5.5.1-12 Strain History of NDL2

Fig.5.5.1-13 Relationship of Strain versus Scan Number of NDL2

Fig.5.5.1-14 Strain History of SDL6

Fig.5.5.1-15 Relationship of Strain versus Scan Number of SDL6

Fig.5.5.1-16 Strain History of SDU5

Fig.5.5.1-17 Relationship of Strain versus Scan Number of SDU5

Fig.5.5.1-18 Strain History of SDU8

Fig.5.5.1-19 Relationship of Strain versus Scan Number of SDU8

C. Strain History of Horizontal Bars

The strain histories of the horizontal bars were shown in Fig.5.5.1-20 to Fig.5.5.1-24. It can be seen from Fig.5.5.1-20 that the horizontal reinforcement in

the range of the diagonal bars near the foundation did not yield, which meant that at any section in the range of the diagonal bars near the foundation, shear force was resisted by the horizontal reinforcement and the diagonal bars simultaneously.

Fig.5.5.1-21 and Fig.5.5.1-22 indicated that the horizontal bars at this height level exerted their strength completely. Fig.5.5.1-23 and Fig.5.5.1-24 show that the horizontal bars in the upper wall panel did not yield, which implied that shear resistant requirement was smaller in the upper wall panel in precast wall compared to that in the integral prototype. With increasing displacement level, the critical horizontal section was just above the diagonal bars near the foundation where fracture of the horizontal reinforcement occurred.

Fig.5.5.1-20 Relationship of Strain versus Scan Number of NH11

Fig.5.5.1-21 Relationship of Strain versus Scan Number of NH10

Fig.5.5.1-22 Relationship of Strain versus Scan Number of NH9

Fig.5.5.1-23 Relationship of Strain versus Scan Number of SH5

Fig.5.5.1-24 Relationship of Strain versus Scan Number of SH3

5.5.2 Load Transfer

In the early loading stages, the diagonal cracks from the bottom corner to the

opposite top corner were the main features of cracking patterns as shown in Fig.4.2.4-5. The diagonal concrete strut played an important role in resisting the lateral load. With increasing load, the diagonal cracks from the lower corner to the opposite higher corner of each wall panel appeared in the upper and the lower panels, which could be seemed as a sign of the changes of the internal load paths. At the same time the upper and the lower wall panels performed as independent with special feature of cracking patterns of squat walls.

Because no evidence of deformation of the horizontal connection was observed, the lateral load could be transferred successfully through the horizontal connection to the lower wall panel. Referring to cracking patterns of the wall and the strain in the horizontal reinforcement at different stages, it can be assumed that in the early stages, before reaching a ductility of 2, the main lateral resistant mechanism was the diagonal concrete strut from the lower corner to the opposite upper corner of W5 as shown in Fig.5.5.2-1. After ductility passed 2, the upper wall panel acted as a squat wall with a low aspect ratio, a diagonal concrete strut starting from the center of top beam appeared. The performance of the upper wall panel influenced the load paths in the whole specimen. As discussed in previous sections, the connection zone can be seen as a continuum through which internal forces in the upper wall panel can be transferred without deviation.

Fig.5.5.2-1 Cracking direction in lower corner

5.5.3 Concrete Crushing and Breaking of Horizontal Bar

When ductility reached 3, the width of concrete bands between diagonal cracks was about 100mm. These diagonal cracks were evenly distributed in the upper and lower wall panels. With increasing displacement levels, two wide cracks just above the diagonal bars near the base intersected to form a “V” shape with about 120 deg. At the same time, many short cracks appeared along the inner boundaries of the I-Beams in the lower wall panel, which intersected with the flexural and diagonal cracks. These short cracks were not observed in the upper wall panel. In late loading cycles, the diagonal cracks aligned with diagonal bars near the base became wider and simultaneously horizontal reinforcement crossing through these cracks yielded. Diagonal cracks from opposite direction intersected these wide diagonal cracks in lower wall panel. The short cracks along the inner boundaries of I-Beams cut

diagonal cracks and flexural cracks at same positions. The density of the cracks increased. Under cyclic load, these small concrete pieces fell down, which reduced nodal area of concrete struts connecting these locations. In the loading course, it can be observed that the diagonal concrete bands resisting the lateral load be sheared one by one at the height of the top end level of the diagonal reinforcement near the foundation. For example, when a positive load was applied, the diagonal concrete bands at this level were sheared from right to left one by one. It can be seen as the course of the failure of the section. Two triangular zones formed at near the up ends of diagonal bars at height level 650mm from base beam. It meant that the percentage of the contribution of the concrete struts to the lateral load dropped and the requirements of the horizontal reinforcement increased. Finally two horizontal bars were fractured with a sudden failure of the specimen.

Fig.5.5.3-1 Features of Diagonal Cracks in Lower Wall Panel

Fig.5.5.3-2 Fracture of Horizontal Bar

Fig.5.5.3-3 Failure Mode (Rear View)

5.5.4 Strut-and-Tie Model

Specimen W5 performed a similar behavior with specimen W4 in the early loading stages. The diagonal reinforcement placed near the foundation did not yield, which implied that the contribution of diagonal bars to lateral load was insignificant. Therefore strut-and-tie models shown in Fig.5.5.4-1 and Fig.5.5.4-2 represented two individual mechanisms to resist lateral load. Based on theory of minimum strain energy, a combined strut-and-tie model for specimen W5 is shown in Fig.5.5.5-3.

After reaching a ductility of 2, cracking patterns indicated that the internal load paths through which the lateral load was transferred to the foundation. The diagonal cracks from the lower corner to the opposite upper corner in each wall panel occurred, which indicated that two wall panel behaved as two independent squat walls. As shown in Fig.5.2.3-2, due to the low aspect ratio of the upper wall panel, two diagonal concrete struts were assumed. According to the internal load paths shown in Fig.5.2.3-2, the strut-and-tie model shown in Fig.5.5.4-4 was developed. The contribution of the diagonal reinforcement located near the foundation was considered in Fig.5.5.4-5. In the whole course of loading, evidence of deformation and damage of the horizontal connection was not observed. Therefore, a solid continuum was assumed to represent the connection zone which was defined in previous sections. In accordance to the theory of minimum strain energy, the total lateral load was distributed to the two load-resistant mechanisms as shown in Fig.5.5.4-6 and Fig.5.5.4-7. The combined strut-and-tie model was shown in Fig.5.5.4-8 which combined the strut-and-tie models shown in Fig.5.5.4-6 and Fig.5.5.4-7. In the model shown in Fig.5.5.4-8, concrete strut BF was not but GL was the main component which transferred the lateral load to foundation. The concrete strut HI took an important role in transferring load from upper wall panel to the lower panel, which caused yielding of the horizontal tie KI. When overstrength of the vertical steels was considered, requirement of tie KI was

increased. Under cyclic loads, due to concrete spalling at upper ends of the diagonal reinforcement near the foundation, the nodal area of the concrete in compression at point I was decreased, which enlarged obligation of the horizontal tie KI to resist the lateral load. At this moment, KI had come into plastic deformation and lost its ability to bear the additional lateral load. As a result, a fracture of the horizontal reinforcement occurred as observed in experiment.

$$T_1 = 2050 \times 322.16 = 660.4kN$$

$$T_2 = 384.5 \times 2 \times 5 \times 78.5 = 301.8kN$$

$$T = T_1 + T_2 = 962.2kN$$

$$d = \frac{T}{0.85 f_c' b} = \frac{962.2 \times 1000}{0.85 \times 34.02 \times 140} = 238mm$$

$$w_s = d' \sin \alpha = 316 \times \sin 41.5^\circ = 209.4mm$$

Distance of center of gravity of vertical bars in tension to the center of compression zone:

$$1800 - 710 - 144 = 946mm$$

$$w_s = d' \sin \alpha = 316 \times \sin 41.5^\circ = 209.4mm$$

$$C_{DB} = 0.75 f_c' b w_s = 0.75 \times 33.5 \times 140 \times 209.4 = 763.6kN$$

Table 5.5.4-1 Strain Energy in strut-and-tie of specimen W5

Mechanism	Member	Force	Length	ϵ	Strain Energy	Contribution Rate
First	CD	87.9	1390	$0.55 \epsilon_y$	$67.2 \epsilon_y$	0.501
	DA	160.6	1150	ϵ_y	$184.7 \epsilon_y$	
	DE	100	1581	ϵ_y	$158.1 \epsilon_y$	
	$\sum F_i l_i$				$410 \epsilon_y$	
Second	CA	160.6	2540	ϵ_y	$408 \epsilon_y$	0.499

Fig.5.5.4-1 Strut-and-Tie Model

Fig.5.5.4-2 Strut-and-Tie Model

Fig.5.5.4-3 Combined Strut-and-Tie Model

Table 5.5.4-2 Strain Energy in strut-and-tie of specimen W5

Mechanism	Member	Force	Length	ε	Strain Energy	Contribution Rate
First	AG'	160.7	800	ε_y	$128.6 \varepsilon_y$	0.48
	CD	133.6	1290	ε_y	$172.3 \varepsilon_y$	
	BE	100	1581	ε_y	$158 \varepsilon_y$	
	BG'	86.8	650	$0.5 \varepsilon_y$	$28.2 \varepsilon_y$	
	G'I'	50	1600	$0.5 \varepsilon_y$	$40 \varepsilon_y$	
	$\sum F_i l_i$				$527.2 \varepsilon_y$	
Second	AG	181.4	650	ε_y	$117.9 \varepsilon_y$	0.52
	GN	118	850	$0.65 \varepsilon_y$	$65.2 \varepsilon_y$	
	NH	83.9	1040	$0.46 \varepsilon_y$	$40.1 \varepsilon_y$	
	GK	78	1450	$0.78 \varepsilon_y$	$88.2 \varepsilon_y$	
	LK	31	919	ε_y	$28.5 \varepsilon_y$	
	KI	100	131	ε_y	$13.1 \varepsilon_y$	
	NM	100	1250	ε_y	$125 \varepsilon_y$	
	$\sum F_i l_i$				$478 \varepsilon_y$

Fig.5.5.4-4 Strut-and-Tie Model

Fig.5.5.4-5 Strut-and-Tie Model

Fig.5.5.4-6 Strut-and-Tie Model

Fig.5.5.4-7 Strut-and-Tie Model

Fig.5.5.4-8 Strut-and-Tie Model

5.6 Behavior of Specimen W6

Specimen W6 is one of the precast counterparts of specimen W4. In specimen W6, two shear connectors were used to transfer the horizontal force in the connection.

5.6.1 Strain History

A. Strain Profiles of Vertical Bars

The strain profiles of the vertical bars for the positive and negative loads were shown in Fig.5.6.1-1 and Fig.5.6.1-2. It can be seen from the figures that in the first four cycles the distribution of the strain in the vertical bars along the wall-foundation interface exhibited almost linear feature, especially in first cycle.

The increased displacement level caused the specimen to enter the plastic deformation stage, as nonlinear feature of the strain profiles was observed. With increasing lateral load, after cracking occurred in the wall, depth of compression zone increased from Fig. 5.6.1-1 and Fig. 5.6.1-2.

Fig.5.6.1-1 Strain Profiles of Vertical Bars for Positive Load

Fig.5.6.1-2 Strain Profiles of Vertical Bars for Negative Load

B. Strain History of Diagonal Bars

When reaching a ductility of 2, the diagonal reinforcement yielded as shown in Fig.5.6.1-3 and Fig.5.6.1-4. In the elastic loading cycles, strain readings of the diagonal bars near the foundation showed that stress in the diagonal bars was low. Therefore the contribution of the diagonal reinforcement to the lateral load resistance was insignificant.

Fig.5.6.1-3 Strain History of Diagonal Bar SD3

Fig.5.6.1-4 Strain History of Diagonal Bar SD4

5.6.2 Strut-and-Model

In the early loading stages, before reaching a ductility of 1, specimen W6 behaved similarly to specimen W5. The cracking patterns were also similar as those of specimen W5. The diagonal cracks from the bottom corner of the wall to the top corner. Strain readings of the diagonal reinforcement near the foundation showed that the diagonal bars did not yield. Similarly to the diagonal reinforcement in specimen W5, the contribution of the diagonal reinforcement near the foundation to the lateral load was not taken into account. Therefore the strut-and-tie model shown in Fig.5.5.4-3 represented the internal forces in the wall in this stage.

5.6.3 Failure Mode

When the displacement level was increased to ductility 2, connection failure occurred with shear failure of the bolts in flexural connectors at right end of the

horizontal connection as shown in Fig.5.6.3-1 and Fig.5.6.3-2. The flexural connection caused fracture of the diagonal bars welded on shear connection. This might be due to construction defect.

Fig.5.6.3-1 Connection Failure in Specimen W6

Fig.5.6.3-2 Shear Failure of Bolt in Flexural Connection

5.7 Summary

Six specimens, two integral prototypes and four precast counterparts, were

tested to failure at Nanyang Technological University. Records of strain gauges placed on horizontal, vertical and diagonal reinforcement and steel sections were analyzed. Considering the cracking patterns and records of strain gauges, strut-and-tie models were developed to represent the internal force paths and explain failure causes.

1. In the early loading stages, the diagonal reinforcement near the wall-foundation interface did not reach their yielding strength in all specimens. In developing strut-and-tie models for these stages, the contribution of the diagonal bars to lateral load acting on the top beam is insignificant, therefore, no strut and tie is used to represent them.
2. In later loading stages, all of the diagonal bars placed near the wall-foundation interface yielded. Their contribution to lateral load should be taken into account.
3. In testing integral walls, W1 and W4, in later loading cycles, a wide crack crossed the wall along the wall-foundation interface. However, such a crack was not observed in the precast walls.
4. In early loading stages, no evidence of deformation and damage was observed in the horizontal connections of the precast walls. The cracking patterns of the precast walls were similar to those of their prototypes.
5. When the displacement level reached a ductility factor of 2, the cracking patterns in the precast walls differed from those of their prototypes. Diagonal cracks started from the lower corner to the opposite upper corner in each wall panel, which made the two wall panels act as independent squat walls. Due to the low aspect ratio of the upper wall panel, two diagonal concrete struts were assumed. And force paths in the upper wall panel influenced force paths in the lower wall panel.
6. Except for the horizontal connection in specimen W6, for which the failure

may have been due to construction defect, the other horizontal connection did not fail during the course of testing. It means that the horizontal connections in this research were alike to successfully transfer the shear between two wall panels.

7. Strut-and-tie models were developed for different loading stages. These strut-and-tie models were used to explain the internal force paths in these specimens and their failure mode and can be used to direct design.