Seismic performance of RC structural squat walls with limited transverse reinforcement

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**SEISMIC PERFORMANCE OF
RC STRUCTURAL SQUAT WALLS WITH
LIMITED TRANSVERSE REINFORCEMENT**

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ABSTRACT

In the last three decades, extensive research works has been conducted to assess the validity of the design provisions of EC8 and ACI 318 for cyclic shear in reinforced concrete (RC) structural walls with low aspect ratios. Significant progress has been achieved in the understanding of global and local responses of such RC structural walls. These squat walls are usually detailed according to current provisions and reinforced against the shear by either conventionally or by adding additional cross-inclined bidiagonal bars to achieve the full ductile behavior of the RC member. Previous research, however, does not provide adequate and conclusive information of structural squat walls with limited transverse reinforcement in boundary columns. Such RC walls may exhibit only limited ductility and the sliding shear mode may dominate. Current research is, therefore, initiated by the need to provide useful and conclusive information related with the local and global responses of squat RC walls with limited transverse reinforcement. In this study, both experimental and numerical investigations of local and global responses of such RC walls under cyclic loadings have been presented in detail.

An experimental program has been carried out to explore the local and global responses of a total of eight squat RC structural walls with limited transverse reinforcement. The global and local behavior of these RC walls from the experiments carried out is described in detail. The influence of several design parameters such as axial compression loads, transverse reinforcements in the wall boundary columns and the presence of construction joints at the wall base, on the behavior of such RC walls under cyclic loadings is also reported herein. Reasonable strut-and-tie models for RC structural walls with and without axial loads are then developed to aid in better understanding the force transfer mechanism and contribution of reinforcement in RC walls based on the experiments carried out.

Next, an analytical approach, combining the inelastic flexure and shear components of deformation, is proposed to properly evaluate the initial stiffness of tested RC walls. The use of this analytical approach which includes a comprehensive parametric study with 180 combinations is carried out with the emphasis on four main parameters that influence wall stiffness: yield strength of outermost longitudinal bars, axial loads, aspect ratios and longitudinal reinforcement content in wall boundaries. These four parameters are studied in detail and a simple expression based on this study is proposed to determine the initial stiffness of squat RC walls as a function of three factors: yield strength of the outermost longitudinal reinforcement, applied axial compression and wall aspect ratios. Comparing with other stiffness prediction expressions, it is shown that the initial stiffness determined by the proposed expression compares better with the experimental results.

Finally, to aid in better understanding of global responses of such RC structural walls, a nonlinear finite element analytical procedure for squat RC structural walls under cyclic loadings is developed and verified with the experimental results. Global responses such as strength capacity, stiffness characteristics and energy dissipation capacity etc. under reversed seismic loadings are described in detail. Moreover, the influence of several paramount parameters such as axial loads, longitudinal reinforcements in the wall boundary elements, aspect ratio, area of boundary columns and the presence of construction joints at the wall base on the global behavior of squat RC walls is also investigated in detail by means of the proposed analytical procedure.

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To my family

CHAPTER ONE

INTRODUCTION

1.1 General

Singapore and Peninsular Malaysia are in a seismic risk area with an active earthquake belt comprising the Sumatra Fault and the subduction zone located at about 350 km away from its closest point. Although there has never been any earthquake damage to this region, ground tremors have been felt many times, and the incidents have increased significantly in number over the last three decades. Strong tremors were felt in buildings in Singapore from the recent north Sumatra Earthquake. In these low to moderate seismic regions like Singapore and Peninsular Malaysia, reinforced concrete (RC) structural walls are normally designed in accordance with the British Standard: BS 8110 which excludes the influence of seismic loading. Hence, structural walls are usually detailed non-seismically or detailed only to provide limited seismic resistance. As such, the wall boundary elements contain a lack of confinement reinforcement and are liable to extensive damages under earthquake excitation due to excessive shear deformation and severe strength degradation. Therefore, it is of great concern that the structural performance of these RC walls may not be adequate to sustain earthquake-induced loads in regions of low to moderate seismicity like Singapore and Peninsular Malaysia. Experimental and analytical studies of RC walls with limited transverse reinforcement are therefore needed to explore their structural performance in terms of strength and deformation characteristics, secant stiffness and energy dissipation capacity.

In the field of experimental studies, extensive research have been conducted in recent years to assess the validity of current design provisions such as ACI 318 and NZS 3101 for cyclic shear in squat RC structural walls. Usually, these walls are detailed according to the current provisions and aimed at achieving its full ductile behavior. However, in low to moderate seismicity regions such as Singapore and Peninsular Malaysia, the ductility demands may not be the same as those required in higher seismicity zones. In such situations, less ductility demand can be expected and thus the required quantities of reinforcement, especially the transverse reinforcement in web and boundary element, can be reduced. Therefore, it is considered to be necessary to understand the seismic performance of squat RC structural walls with limited transverse reinforcements located in these low to moderate seismicity regions. However, up to date only a few experimental investigations [M1, P2, Y1] of the behavior of such RC structural walls under cyclic loadings have been conducted and from these investigations it is found that there is still insufficient information regarding the structural performance such as available ductility and energy dissipation capacity of such RC walls with axial compression. Moreover, current available experimental data related to the behavior of squat RC walls with weak interface like construction joints are rather inconclusive. To provide adequate and conclusive information, experimental studies for such RC squat walls under axial compression and the presence of construction joints should be carried out to investigate their seismic performances.

In the field of analytical investigations of such RC walls, firstly it is found that previous research in initial wall stiffness evaluation are either over-simplified which may lead to inaccurate assessment of the wall stiffness or initial wall stiffness which can only be justified for slender structural walls whose response is dominated by flexure. Thus, proper stiffness evaluations applied to RC structural walls with low aspect ratios as

their response may be controlled by shear deformations should be carried out to improve wall stiffness predictions. Secondly, numerous analytical models in the past decades were developed in the modeling of RC structural walls to explore their global responses. Most of these were macroscopic models for RC structural walls due to their easy application and in these studies great success has been achieved at the element level. However, these analytical results are usually valid only for the specific conditions upon which the derivation of the model is based upon. Moreover, most of the previous work in the finite element analysis of the behavior of RC walls concentrated on the development of the material models that could reproduce experimental results and few research studies used the finite element method to investigate behavior of RC walls other than that of the specimens tested in the laboratory. Therefore, a nonlinear finite element analytical procedure incorporating general-purpose microscopic models is needed to be developed to examine the structural performances of squat RC structural walls under seismic loadings. An extensive parametric study including several critical parameters: axial loads, longitudinal reinforcements in the wall boundary elements, aspect ratio, area of boundary columns and the presence of construction joints at the wall base are also considered to be necessary to investigate their influence on the seismic performance of such RC walls.

1.2 Objectives of Present Research

The main objectives of the present study for squat RC structural walls with limited transverse reinforcement are to compile information to:

- examine the structural performance in terms of the displacement and strength

capacity, curvature distribution, the secant stiffness degradation and the energy dissipation characteristics of RC structural walls with limited transverse reinforcement.

- report the influence of several design parameters such as axial loads, transverse reinforcements in the wall boundary elements, and the presence of construction joints at the wall base on the seismic behavior of RC walls tested.
- develop reasonable strut-and-tie models to aid in better understanding the force transfer mechanism and contribution of reinforcement in RC walls tested.
- evaluate stiffness characteristics of squat RC structural walls with limited transverse reinforcement.
- use a nonlinear finite element analytical procedure to aid in better understanding the global responses of squat RC structural walls under several critical parameters such as axial loads, longitudinal reinforcements in the wall boundary elements, aspect ratio, area of boundary columns and the presence of construction joints at the wall base.

1.3 Outline of the Report

The introduction to and objectives of this study is presented in this Chapter.

In Chapter 2, structural performances of five low-rise RC walls with limited transverse reinforcement and an aspect ratio of 1.125, are examined by subjecting them to low

levels of axial compression and cyclic lateral loading. The influence of axial loads, transverse reinforcements in the wall boundary elements, and the effect of the presence of construction joints at the wall base on the seismic behavior of walls are also reported in this chapter. Towards the end of the chapter, reasonable strut-and-tie models are developed to aid in understanding the force transfer mechanism and contribution of reinforcement in walls tested.

In Chapter 3, three medium-rise RC structural walls with an aspect ratio of 1.625 are tested to examine the structural performances of RC walls with limited transverse reinforcement. The effect of axial loads, transverse reinforcements in the wall boundary elements, and the effect of the presence of construction joints at the wall based on the seismic behavior of walls are also reported from this chapter. Towards the end of the chapter, reasonable strut-and-tie models are developed to aid in understanding the force transfer mechanism and contribution of reinforcement in walls tested.

In Chapter 4, an analytical approach, combining inelastic flexure and shear components of deformation, is proposed to properly evaluate the initial stiffness of the RC walls tested. According to this verified analytical approach, an extensive parametric study including a total of 180 combinations is carried out to investigate the influence of several critical parameters on the initial stiffness of RC walls. Towards the end of the chapter, a simple expression based on this parametric study is proposed to determine the initial stiffness of walls as a function of three factors: yield strength of the outermost longitudinal reinforcement, applied axial compression and wall aspect ratios.

In Chapter 5, a nonlinear finite element analytical procedure is provided to aid in better understanding the global responses of squat RC structural walls with limited transverse

reinforcement. The global responses such as strength capacity, deformation characteristics, energy dissipation capacity and equivalent damping of such structural walls under reversed seismic loadings are described and investigated in detail. Moreover, the influence of several paramount parameters such as axial loads, longitudinal reinforcements in the wall boundary elements, aspect ratio, area of boundary columns and the effect of the presence of construction joints at the wall base on the seismic behavior of walls are also investigated by means of the developed finite element models.

In Chapter 6, conclusions are drawn for this study and recommendations are presented for future work.

CHAPTER TWO

SEISMIC PERFORMANCE OF LOW-RISE STRUCTURAL WALLS WITH LIMITED TRANSVERSE REINFORCEMENT

Abstract

The main objective of this study was to study the available ductility of low-rise reinforced concrete (RC) walls containing less confining reinforcement than that recommended by the New Zealand Concrete Design Code [N1] and American Concrete Institute Code [A1]. Five RC walls, with an aspect ratio of 1.125, were tested by subjecting them to low levels of axial compression loading and cyclic lateral loading which simulated a moderate earthquake to examine the structural performance of the walls with limited transverse reinforcement. Conclusions are reached concerning the displacement capacity, strength capacity, curvature distribution, the secant stiffness degradation and the energy dissipation characteristics shown by the walls on the seismic behavior with limited transverse reinforcement. The influence of axial loads, transverse reinforcements in the wall boundary elements, and the presence of construction joints at the wall base on the seismic behavior of walls are also reported in this study. Towards the end of the chapter, reasonable strut-and-tie models are developed to aid in understanding the force transfer mechanism and contribution of reinforcement in walls tested.

Keywords: Low-rise structural walls; Deformation capacity; Seismic performance; Low to moderate seismic; Strut-and-tie model

2.1 Introduction and Background

RC structural walls are frequently used in buildings primarily to resist lateral loads imposed by wind and earthquakes. The superior performance of buildings containing structural walls in resisting earthquakes was well documented by Fintel [F1]. In recent years, extensive research [L1, M2, P1-5, T1-2, and W1] have been conducted to assess the validity of current design provisions [A1-2, C1, and N1] for cyclic shear in squat reinforced concrete (RC) walls. Usually, these walls are detailed according to the current provisions and aimed at achieving full ductile behavior of structural walls. Presently, various countries have recommendations for confining reinforcement to ensure that the required ductility demand can be achieved. This is in light of design equations for the amount of confining reinforcement, the limitation of stirrup spacing, and the length of confined regions. However, in low to moderate seismicity regions such as Singapore and Malaysia, the ductility demands may not be the same as that in higher seismicity zones. In such situations, less ductility demand can be expected and therefore the required quantities of reinforcement, especially the transverse reinforcement in web and boundary element, can be reduced. In this research, the seismic performance of five low-rise structural RC walls, which need only limited transverse reinforcements, is of interest.

Besides, many wall structures before the capacity design procedures were introduced may possess considerable amount of inherent lateral strength which is excess of that predicted for fully ductile systems. This suggests that the ductility demand in such “strong” structures will be less and a trade-off between the ductility and strength should be considered. In practice, such type of walls should not be considered to be dangerous during earthquakes as long as the shear strength of the walls is kept greater than that needed for a fully ductile structure and energy dissipation is considered to be acceptable. As such, the potential strength of

structural walls is in excess of that required when considering fully ductile response to the design earthquake. Thus, it is important to identify the strength capacity, ductility capacity and energy dissipation capacity of such type of walls when displaced its elastic limit.

Currently, although several experimental investigations [G1, M1, P2, Y1] of the behavior of low-rise RC walls with limited transverse reinforcements under simulated seismic loading have been conducted, there is still insufficient information regarding the available ductility of walls with axial compression. Moreover, current available experimental data related to the behavior of squat reinforced concrete walls with weak interface like construction joints are rather inconclusive. The study was motivated by the need to better understand the basic behavior of such walls. Conclusions are reached concerning the displacement capacity, strength capacity, curvature distribution, the secant stiffness degradation and the energy dissipation characteristics shown by the seismic behavior of walls with limited transverse reinforcement.

The present study for low-rise structural walls with limited transverse reinforcements compiles information for the economical design of structures which fall between full ductility and elasticity: structures with strengths greater than that required by seismic loading for fully ductile behavior, or less important structures which do not warrant detailing for full ductility. Besides, this comprehensive experimental program can present better understanding on the basic behavior of limited ductile structural walls with the presence of axial loadings, various quantities of transverse reinforcements at boundaries, and the presence of construction joints. Finally, the proposed strut-and-tie models can offer insights into the concept of shear transfer and the contribution of reinforcement in reinforced concrete squat walls.

2.2 Experimental Program

The five low-rise structural walls, referred to as Specimens LW1-LW5, were constructed and tested as isolated cantilever walls with an aspect ratio of 1.125. The experimental program presented herein is aimed at investigating the performance of squat reinforced concrete walls with limited transverse reinforcement. The effects of reinforcement detailing, axial load and construction joints at wall base on strength capacity, stiffness characteristics, and energy dissipation capacity of walls with limited transverse reinforcement were investigated. Based on this, the experimental procedure includes the following objectives:

- (1) The basic performance such as the strength capacity, available ductility, and energy dissipation capacity of the prototype walls should be the main concern of this research;
- (2) The effects of main test parameters on the performance of the structural walls with limited transverse reinforcement;
- (3) The load path and crack patterns should be presented at each ductility/drift level to study the shear transfer mechanisms and failure modes;
- (4) Deformations of the test specimens due to shear, flexure, and sliding should be measured to investigate mechanisms of shear and flexure contributions.

2.2.1 Material Properties

Ready mixed concrete with 13 mm maximum aggregate specified by a characteristic strength of 35 MPa was used to cast the specimens. A total of thirty-three 150×150×150 mm cubes and 150×300 mm cylinders were cast and tested according to the standard procedures. The average concrete cylinder compressive strength f'_c for different specimens observed after 28 days was varied between

32.43 Mpa and 38.81 Mpa.

Two types of steel bars, high yield steel bar (T bar) with the nominal yield strength of 460 Mpa and mild steel bar (R bar) with the nominal yield strength of 250 Mpa, were used in all specimens. Fig. 2.1 displays the typical stress-strain relationships of the bars. Among all bars, T10, R10, and R6 were used in the walls while T13 and T20 were used at the top and base beams.

2.2.2 Code Provisions for Confining Reinforcement in Plastic Hinge Regions

Due to the existence of different requirements for the amount of confining reinforcement in walls in the ACI 318 [A1] and NZS 3101 [N1] codes, design equations in both design codes to ensure adequate ductility are described as follows.

2.2.2.1 ACI 318 Code Provisions

The required area of hoop reinforcement is given by the larger of

$$A_{sh} = 0.3 s h_c \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f_c'}{f_{yh}} \quad (2.1)$$

and

$$A_{sh} = 0.09 s h_c \frac{f_c'}{f_{yh}} \quad (2.2)$$

where A_{sh} is the total cross-sectional area of transverse confining reinforcement within spacing s and perpendicular to dimension h_c ; s is the spacing of transverse reinforcement measured along the longitudinal axis of the structural member; h_c is the cross-sectional dimension of column core measured

center-to-center of confining reinforcement; A_g is the gross area of section; A_{ch} is the cross-sectional area of a structural member measured out-to-out of transverse reinforcement;

2.2.2.2 NZS 3101:1995 Code Provisions

Where the neutral axis depth in the potential yield regions of a wall, computed for the approximate design forces for the ultimate limit state, exceeds:

$$c_c = (0.3\phi_0 / \mu)L_w \quad (2.3)$$

where c_c is the distance of the critical neutral axis from the compression edge of the wall section at the ultimate limit state; ϕ_0 is the ratio of moment of resistance at overstrength to moment resulting from specified earthquake forces, where both moments refer to the base section of wall. μ is the displacement ductility capacity relied on in the design; L_w is the horizontal length of wall.

The following requirements of the transverse reinforcing steel shall be satisfied in that part of the wall section which is subjected to compression strains due to the design forces.

$$A_{sh} = \left(\frac{\mu}{40} + 0.1 \right) s_h h_c \frac{A_g}{A_c} \frac{f_c'}{f_{yh}} \left(\frac{c}{L_w} - 0.07 \right) \quad (2.4)$$

2.2.3 Application of Design Procedures

In the following, the flexure and shear design strategy for structural walls of limited

transverse reinforcements is presented in succession. As a general rule, the test walls are proportioned to ensure an eventual flexure rather than shear failure, that is to provide flexure strength less than the shear strength. The flexure strength of the specimen is calculated by means of the conventional sectional analysis, taking into account the effect of reinforcement hardening. The target for shear design of walls is to prevent wall shear failure by providing shear resistance greater than the shear demand. At the design stage, the shear demand is unknown, thus the designer tries to provide shear supply greater than the factored design shears which is an estimate of the shear demand and obtained from the results of earthquake response analysis. In this case, prior to the application of demand analysis, the required lateral force is given by Eq. (2.5) in terms of the pre-assumed displacement ductility capacity, μ_Δ , of 3 which is the lower-bound value recommended by NZS 3101 code [N1] for the fully ductile walls.

$$V_u = \omega_v \phi_{0,w} V_E \leq \mu_\Delta V_E / \phi \quad (2.5)$$

where V_E is the shear demand derived from code-specified lateral static forces for a given displacement ductility; ω_v is the dynamic shear magnification factor and $\phi_{0,w}$ is the over-strength factor for a wall. This equation indicates that the ideal shear strength of the wall need not be taken larger than that corresponding to elastic response because limited inelastic deformations of the walls can be expected for walls with limited transverse reinforcement. It is a conservative limit which is based on the “equal-displacement concept”. However, due to the presence of short period for the assumed walls, the “equal-energy concept” should be applied as a more practical limit. In this study, the reference wall is designed to resist a shear force of $V_E = 320$ kN according to a displacement ductility capacity of $\mu_\Delta = 3$.

For the determination of the required amount of shear reinforcement, the contribution of concrete in the total shear resistance should be estimated. As seen in the literature review, there existed a large number of empirical equations for the contribution of concrete to shear strength, each with its own range of applicability. However, none of the equations apply precisely to the present case, low-rise walls with limited transverse reinforcements under seismic loading. Thus, the calculation of the ideal shear strength for the present test units was somewhat undefined. In this case, the shear strength of low-rise walls is evaluated by the method proposed by ACI 318 [A1] or NZS 3101 [N1]. As stipulated in these codes, the shear strength of low-rise walls is calculated by the sum of the shear contributions from the concrete and web shear reinforcements. Represented by the concrete shear stress, the contribution of concrete [N1] is taken as lesser of following:

$$v_{uc} = 0.27\sqrt{f'_c} + \frac{N_u}{4A_g} \quad (2.6)$$

or

$$v_{uc} = 0.05\sqrt{f'_c} + \frac{l_w(0.1\sqrt{f'_c} + 0.2\frac{N_u}{A_g})}{\left(\frac{M_u}{V_u} - \frac{l_w}{2}\right)} \quad (2.7)$$

where N_u is the axial load (negative for tension), and M_u, V_u are the moment and shear force respectively at the section at the ultimate limit state.

Based on the modified truss analogy and by assuming the strut angle to be 45° , the contribution of web transverse reinforcement at a spacing of s is given by

$$V_{us} = \frac{A_t f_{yt} d_w}{s} \quad (2.8)$$

where A_t is the area of transverse reinforcement within a distance s , d_w is the effective length of wall and normally set to be 80% of the whole wall length.

The controlled wall LW1, which is designed based on aforementioned considerations, is shown in Table 2.1. All five specimens have the same web reinforcement, consisting of two curtains (orthogonal grids) of 10 mm diameter high yield steel bars spaced at 250 mm. This gives a reinforcement ratio of 0.50 percent. The 150 mm×300 mm boundary elements are reinforced with eight mild steel bars of 10 mm diameter; giving a reinforcing ratio of 1.4 percent (minimum code requirement is 1.0 percent). Reduced confinement in the boundary elements is provided by 6.0 mm diameter closed stirrups (hoops) spaced at 75 mm corresponding to 70% of the transverse confining reinforcement required by NZS 3101 at a limited displacement ductility of 3.0 and 25% of that required by ACI 318 as shown in Eq. (2.4) and Eq. (2.1) respectively. While for Specimen LW3, the vertical spacing of the hoops is meant to be 200 mm which corresponds to 30% and 10% of the transverse confining reinforcement required by NZS 3101 and ACI 318 respectively for seismic detailing of fully ductile walls. The overall dimensions and reinforcement details of the controlled specimen LW1 are shown in Fig. 2.2. Each low-rise wall specimen in this test was 2000 mm wide, 2000 mm high and 120 mm thick. This provided the value of aspect ratios for all five specimens with 1.125 which was calculated according to the respective wall height of 2250 mm measured by the vertical distance between the lateral loading point and the wall base.

The specimens were cast monolithically in the vertical direction except that for Specimens LW4 and LW5 with construction joints at the base, they were kept to stand for three days after the concrete for the base beam had been poured, vibrated, and leveled off. Just before the upper part of concrete was poured, the hardened concrete and the reinforcing bars in the construction joint area were brushed to

remove any loose particles. Then, the base beam concrete surface was moistened and the fresh concrete was poured into the upper part of the moulds. After seven days, the moulds were removed and the specimens were allowed to expose to laboratory environment until just before testing. In the end, a day prior to testing the outer surface of every specimen was made extra smooth for drawing crack patterns during testing.

2.2.4 Experimental Set-up and Loading History

The test rig used in this study is shown in Fig. 2.3. It consisted of two main independent systems: an in-plane loading system and an in-plane base beam reaction system. The in-plane loading system comprises one horizontal hydraulic actuator which was fixed to the reaction wall and two vertical actuators connected to the strong floor. The test units were subjected to in-plane, reversed cyclic loading from the horizontal double-acting actuator applied at the level of the top steel transfer beam (transfer beam 2) as shown in Fig. 2.3. The hydraulic actuator with 1000 mm stroke possessed a capacity of 1000 kN in compression and tension. The axial loading was applied through two vertical actuators, each with a compression capacity of 1000 kN and 500 mm stroke, attached to the top beam system, as shown in Fig. 2.3. Fig. 2.3 also shows two steel transfer beams, transfer beam 1 and transfer beam 2 which were conservatively designed and built to transfer the lateral loading to the test specimen. It should be noted that high yield steel bolts were provided in the connection and tightened to efficiently transfer the lateral loading to the test specimen. Two levels of constant axial load were adopted in the testing program. They corresponded to 0.0 and 0.05 of the cylinder compressive strength of the wall cross section that is equal to $f_c' A_g$ which was considered representative of the amount of axial load at the base of the wall of a single story and a low-rise building, respectively. After the total constant axial force was applied, the horizontal loading

would be introduced through the top steel beam (transfer beam 2) of the specimens.

The base beam of the specimens was anchored to the laboratory floor with twelve high strength rods that prevent uplifting of the specimens and horizontal sliding of the units along the floor during the application of the horizontal loading. Every high strength rod was pre-stressed to efficiently restrain the rotation and sliding of the specimen during the test. During the experimental set-up, laser point machines were employed to ensure that the center lines of the three actuators and the specimen tested are in the same vertical plane. This is to effectively avoid out-of-plane buckling of walls during testing.

In the previous tests of ductile members [M1], the displacement loading, in which the level of displacement increases according to the ductility factor ($\mu_{\Delta} = \Delta/\Delta_y$), controls the subsequent cycles. However, since the present test units were expected to exhibit only limited ductility, it was thought more valuable to displace the units to deflections corresponding to selected values of drift due to the fact that the ductility factor ($\mu_{\Delta} = \Delta/\Delta_y$) depends heavily upon the definition of Δ_y which is not readily identified. As mentioned in the previous research [M1], a value of $\Delta/h_w = 0.01$ is considered a practical limit on the drift as this value realistically is expected in low-rise structural wall buildings. Herein the end of the test was reached at a drift ratio of 1.0% or the strength dropped to less than 80% of the recorded maximum loading. The test specimen was subjected to two cycles at each displacement level except that only one cycle was applied to the specimen at a drift ratio of 1/2000. Fig. 2.4 demonstrates the detailed loading sequences during the testing. The axial loading, whenever present, would be kept constant during the entire test by applying load control to the vertical actuator. Moreover, hinged connections at the tips of both the vertical and the horizontal actuator prevent any substantial restraint to the rotation of

the top of the wall, thus insuring cantilever behavior.

2.2.5 Instrumentation of Wall Specimens

For measurement of top deflection, flexure deformations and shear deformations, two types of Linear Variable Differential Transducers (LVDT), with 100 mm travel and 50 mm travel, were introduced as shown in Fig. 2.5. One LVDT numbered as L14 with 100 mm travel was installed at the top of the specimen to monitor the top displacement. A total of ten LVDTs were arranged along the two vertical edges of the specimens to measure the flexure deformation. The panel shear deformations were detected by two LVDTs (L4 and L5) distributed along diagonal directions of the panels. Two inclined LVDTs (L2 and L3) with one end of the steel rods at the base beam were used to measure the sliding deformations of the wall panels. The sliding of base beam was detected by one LVDT named as L1 with 50 mm travel positioned at the strong floor.

FLA type 5 mm-gauge length strain gauges with 10 m vinyl-insulated lead wires were used to measure the local strains of the selected reinforcing bars. The strain gauges were attached to merely one layer of the reinforcing nets such as only the outer layer for the bars of boundary elements was considered to attach the strain gauges due to the symmetric configurations of the wall units. During testing, the strains of the bars were recorded by an automatic data-logger, and a strain gauge was deemed no longer reliable when the strain exceeded 0.02.

2.3 Experimental Results of the Tested Specimens

The global behavior, represented by crack patterns and hysteretic loops, and local response such as longitudinal and transverse bar strain distribution of the tested

specimens are presented in the following figures. For those concerning the crack pattern at different drift ratios, it should be noted that the dashed lines in the grids, indicating the spacing of the reinforcement, represent the negative cracks opened during negative loading while the continuous lines refer to the positive cracks in the positive loading. The blackish areas represent the splitting of the concrete.

2.3.1 Experimental Results of Specimen LW1

Unlike the applied loading history of other specimens as shown in Fig. 2.4, the reference wall LW1 as shown in Fig. 2.2 was firstly displaced with one cycle at drift ratios of 0.1%, 0.25% and 0.33% and then it was applied with two cycles at each continued drift ratio of 0.5% or 1.0%. However, such type of loading history was later found to be providing inadequate data for analyzing the seismic performance and that in the other specimens, minor modifications in the loading sequences were made. The modified loading history is illustrated in Fig. 2.4.

2.3.1.1 Global Behavior

Fig. 2.6 demonstrates the crack patterns and failure modes of the reference wall LW1 at drift ratios of 0.1%, 0.5% and 1.0% corresponding to the initial cracking stage, crack development stage and final failure stage, respectively. As shown in Fig. 2.6(a) the initial flexure cracks at the lower part of the reference wall were observed at a lateral displacement of approximately 2.2 mm corresponding to a drift ratio of 0.1%. With increasing lateral displacements, shear cracks propagated from the wall boundaries toward the opposite side and from the bottom upward. Displacements up to 5.0 mm first-yield of flexure reinforcements in wall boundary elements occurred and thereafter a number of web cracks and their apparent inclination to the horizontal increased due to the joining of web cracks with cracks originating higher

up in the boundary elements were observed. Fig. 2.6(b) presented the post-yield crack patterns at a lateral displacement of 11.2 mm corresponding to a drift ratio of 0.5%. By a displacement of 18 mm, the web cracks propagated more extensively toward the opposite side in the lower part of the wall and several vertical cracks also appeared at the boundaries near the wall base which indicated spalling of concrete cover at these locations. At a displacement level of 22.5 mm the concrete in the lower boundary elements of the wall was spalling considerably and the web cracks in the lower part of the wall opened more widely.

The lateral load versus top displacement relationship for wall LW1 is shown in Fig. 2.7. The ideal strength of $V_i = 321$ kN was obtained from rational section analysis by use of computer program RESPONSE-2000 and exceeded by 12% and 2% for the positive and negative loadings, respectively. It was observed almost linear elastic behavior up to a magnitude of base shear of 304 kN (5 mm) which is close to the predicted yielding of the outermost rebars. For the positive loadings, this presented an initial stiffness of 60.6 kN/mm and hereafter the slope of the response curve changed significantly which indicated considerably degradation of the wall secant stiffness. The maximum base shear of 361 kN was achieved at 18 mm top lateral displacement. At this time, a number of diagonal struts were formed and spalling of the concrete cover at the wall base was observed. Increasing the top displacement to 22.5 mm resulted in a reduction of its shear capacity by 17% to a magnitude of 300 kN. At this last stage, it could be seen from Fig. 2.6© that the wall crack pattern was dominated by flexure and hence a flexure - shear failure mode as expected was developed.

2.3.1.1 Local Response

Fig. 2.8 shows the longitudinal strain distribution of the outermost rebar (#A bar)

with the variation of wall drift ratios in three different positions along the wall height: 50 mm, 550 mm, and 1550 mm above the wall base. Longitudinal strains under both positive and negative loading directions are demonstrated. Under the positive loading direction, it is observed that the first yield of outermost bar occurs at a drift ratio of 0.25% and thereafter the strain increases more rapidly. However, in the negative loading direction, the strain of the longitudinal bar under compression is observed to be not yielding. This reduction of the compression strain can be due to the effect of existing residual tensile strain of the longitudinal bars during testing. Under the negative loading direction, the strain distribution of other longitudinal bars along two different sections is also illustrated in Fig. 2.9(a) and Fig. 2.9(b), respectively. The two wall sections, section 1-1 and section 2-2 as shown in Figs. 2.9(a) and 2.9(b), are located at wall heights of 50 mm and 550 mm, respectively above the wall base. As indicated in Figs. 2.9(a) and 2.9(b), with the increase of wall drift ratios, the neutral axis depth for both two wall sections is observed to shift from the section middle to around 200 mm calculated from the left flange. The flexure plane section hypothesis can be applied to wall section 1-1 till a drift ratio of 0.33%, while it is not suitable for wall section 2-2.

Figs. 2.10(a) and 2.10(b) depict the strain distribution with the variation of the wall drift ratios for two different horizontal web bars: R bar and T bar, respectively. The two horizontal bars locate at the wall heights of 250 mm and 750 mm from the base and in each horizontal bar, the strain values obtained from four strain gauges are plotted under both positive and negative loading directions. In general, the strains in the specified horizontal bars seldom reach yielding. In the positive loadings, larger strain values are observed to occur along the diagonal strut (R7, R8 in R bar and T14, T15 in T bar) and under the negative loadings, it is likewise obvious (R5, R6 in R bar and T13, T14 in T bar).

2.3.2 Experimental Results of Specimen LW2

The geometry layout and reinforcement details of specimen LW2 are presented to be the same as that of specimen LW1. The test of specimen LW2 is intended to investigate the effect of low to moderate axial load on the global behavior and local response of low-rise structural walls. Different from that corresponding to specimen LW1, the applied loading history with the increase of wall drift ratios is illustrated in Fig. 2.4.

2.3.2.1 Global Behavior

The crack patterns and failure modes of specimen LW2 subjected to axial loadings are shown in Fig. 2.11. Fig. 2.12 presents the typical hysteretic loops of specimen LW2. As shown in Fig. 2.11, two initial flexure cracks located at the lower part of each wall boundary element were observed at a drift ratio of 0.1% for a base shear of approximately 276 kN. With increasing cycle numbers, the outermost flexure reinforcement in left boundary elements experienced yielding at a drift ratio of 0.2% (4.5 mm) corresponding to a base shear of 400 kN and based on this, a magnitude of yield displacement of 5.9 mm was obtained as shown in Fig. 2.12. For a displacement up to 11.25 mm, the flexure cracks in the boundaries became denser and extended up to the top of the wall. At this stage, almost all shear cracks in the web penetrated into the opposite side of the wall were developed. The maximum base shear of 579.9 kN was achieved at a displacement of 22.68 mm corresponding to a displacement ductility of 3.8 for the positive loadings. With increasing displacements, it was observed that the inclined web cracks opened more widely and formed several diagonal struts apparently.

Final failure occurred with the observed behaviors that the small quantity of

concrete in the right flange near the bottom was spalling and four shear cracks in the lower part of web opened significantly wide. By comparison with the crack patterns of the reference wall LW1, less shear cracks in the web of the wall LW2 was observed due to the presence of the axial loadings which played a beneficial role in controlling wall cracking.

2.3.2.2 Local Response

A typical strain distribution of an outermost longitudinal bar (#A bar) in left wall flange along the wall drift ratios is shown in Fig. 2.13. As indicated in Fig. 2.13, first yielding of the longitudinal bar occurs at a drift ratio of approximately 0.2%. Maximum strain, which is less than that of specimen LW1, is obtained for strain gauge A31 located at 550 mm from the base at ultimate state. This may be induced by the presence of low to moderate axial loads in the wall during testing which inhibits the development of the flexure yielding of longitudinal bars at the base. Figs. 2.14(a) and 2.14(b) present the strain profiles of the vertical bars along section 1-1 and section 2-2, respectively. As can be seen from these two figures, plane section hypothesis can not be applied to wall section 1-1 after the attainment of a wall drift ratio of 0.5% corresponding to the crack development stage.

The strain distribution of two horizontal web bars with the variation of the drift ratios is illustrated in Figs. 2.15(a) and 2.15(b). It can be seen that the strains are affected greatly by the presence of nearby cracks. A rapid increase in the plot generally results from a crack having crossed the horizontal bar at the position of the strain gauge. The stress in the surrounding region of concrete is effectively transferred to the reinforcing bar at the crack.

2.3.3 Experimental Results of Specimen LW3

The main purpose of this testing was to investigate the confinement effect of transverse reinforcements in wall boundaries on the seismic performance of the specimen. For specimen LW3, the vertical spacing of transverse reinforcements in wall boundaries was modified from 75 mm, which was applied in wall LW2, to 200 mm. This led to the confinement reinforcement ratio represented by volumetric ratio of transverse reinforcement in wall boundaries changing from 0.85% to 0.35%.

2.3.3.1 Global Behavior

The crack patterns and hysteretic loops of specimen LW3 are shown in Fig. 2.16 and Fig. 2.17, respectively. Hairline flexure cracks in the boundaries were initiated at a drift ratio of approximately 0.12% corresponding to a base shear of 270.4 kN which is close to that recorded by wall LW2. The first post-cracking patterns of wall LW3 at a drift ratio of 0.17% is presented in Fig. 2.16(a). Cycling the wall to a drift ratio of 0.22%, yielding of the outermost flexure reinforcement in wall left boundary occurred and the recorded base shear at this stage reached at 402.3 kN. Fig. 2.16(b) shows the post-yielding crack patterns of wall LW3 at a drift ratio of 0.5% which were rather similar to those of wall LW2 with the observed behavior that almost horizontally flexure cracks extended up to the top of wall and inclined shear cracks propagated from the wall boundaries towards the opposite side. At that stage in testing, the maximum base shear of 523.8 kN was achieved corresponding to a lateral displacement of 11.25 mm for the positive loadings. When the displacements were further increased to 15 mm, severe deterioration of the concrete within the left wall flange at a wall height of approximately 500-1000 mm was observed and the buckling of flexure reinforcements in this area followed. The final crack patterns and failure modes of wall LW3 are shown in Fig. 2.16(c).

2.3.3.2 Local Response

The plot of strain of the longitudinal bar (#K bar) in the vertical boundary elements of the specimen subjected to both positive and negative loading directions is shown in Fig. 2.18. Strains reach approximately yield strain at a drift ratio of 0.22% just before the attainment of flexure strength. In the positive loading direction, the bar is subjected to compression and the measured strain is less than that under the negative loading direction. In the tensile boundary element, strains are not distributed uniformly or changed rapidly with height above the base level. This mixed pattern of strain distribution is clearly indicative of both the flexure and shear interaction. The strain distribution of the vertical bars in wall flanges and web along the wall sections under the negative loadings is also illustrated in Figs. 2.19(a) and 2.19(b). It is seen that in moving from horizontal section 1-1 (Fig. 2.19(a)) to horizontal section 2-2 (Fig. 2.19(b)), the neutral axis shifts toward the tension face of the wall. At section 1-1, compressive strains at ultimate state are observed within approximately 200 mm ($0.1L_w$) of the compression face. At section 2-2, compressive strains are attained within 500 mm ($0.25L_w$) of the compression face.

The strains along two different selected horizontal web bars at the various drift peaks are shown in Figs. 2.20(a) and 2.20(b). In general, the strains are observed to be small and are concentrated along the main diagonals (R8, T15 for positive loading direction and R6, T14 for negative loading direction).

2.3.4 Experimental Results of Specimen LW4

The specimen LW4 was chosen to study the effect of existing dry construction joints on the seismic behavior of the walls. The overall dimension and reinforcement detailing of the specimen were the same as that of the reference wall LW1 except

that there were dry construction joints at the base of wall LW4.

2.3.4.1 Global Behavior

The crack patterns and failure modes of wall LW4 are shown in Fig. 2.21. It can be seen that the specimen experienced first cracking at a drift ratio of 0.1% for a base shear of approximately 203.5 kN. In light of recorded strain of flexure reinforcements in the boundary elements, the first yielding of those bars was observed at a drift ratio of 0.2% for a base shear of 300.6 kN. With increasing cycle numbers, similar developing patterns of the flexure and shear cracks with those of the reference wall LW1 were observed until the wall LW4 was displaced to a drift ratio of 0.67% corresponding to a maximum base shear of 365.1 kN. After that stage in testing, the flexure cracks at the bottom of the wall interconnected roughly and the sliding shear failure occurred when the lateral displacements of the wall reached at a drift ratio of 1.0%.

By contrast with the wall LW1, the hysteretic loops of the wall LW4 as shown in Fig. 2.22 depict more degree of pinching due to the presence of dry construction joints at the wall base which resulted in a reduction of the sliding shear strength and hence the sliding shear becomes more important.

2.3.4.2 Local Response

Fig. 2.23 plots the strain distribution in the outermost longitudinal bar (#A bar) in vertical flange of specimen LW4. First yield strain is reached at a drift ratio of 0.25% under the positive loading direction. In the compressive flange, strains are small and less than that in the tensile flange for a given drift ratio. After the attainment of drift ratio of 0.5%, it is observed from Fig. 2.23 that almost the whole

bar along the wall height experiences yielding since the ideal flexure strength is attained. Moreover, rapid increase in longitudinal strain at wall height 50 mm (position A29) occurs at a drift ratio of 0.67% which can be induced by the strain gauge crossed by a crack. The strains in other vertical bars of the wall flanges and web along with the wall section 1-1 and section 2-2 are also presented in Figs. 2.24(a) and 2.24(b), respectively. It can be seen that plane section 1-1 does not remain plane after a drift ratio of 0.5%. This indicates an increasing influence of shear on the wall behavior. When there is a change of wall sections from section 1-1 to section 2-2, the neutral axis moves away from the compression face of the wall.

Strains along two different horizontal web bars, R bar and T bar, at the various drift ratios are shown in Figs. 2.25(a) and 2.25(b), respectively. In general, the strains are small and increase with added wall drift ratios. The strains are mainly concentrated along the main diagonals.

2.3.5 Experimental Results of Specimen LW5

For the purpose of investigating the behavior of walls with existing construction joints at the base and at the same time exposed to a medium level of axial loadings, the wall LW5 was tested to make a comparison with the experimental results of the wall LW2 which was subjected to a same level of axial loading but with no construction joints at the wall base.

2.3.5.1 Global Behavior

The crack pattern and final failure mode of the tested wall are shown in Fig. 2.26. During testing, the initial flexure cracks in the lower part of the wall developed at a displacement of 3.25 mm for a base shear of approximately 310.4 kN. Increasing

displacement to 4.8 mm resulted in the yielding of the flexure reinforcements in the boundary elements. Specimen LW5 developed its maximum base shear of 520 kN at 15.39 mm top lateral displacement. At this time, a large portion of the cracks was formed and several diagonal struts crossed in the middle of the wall web. Further cycling the wall LW5 to 22 mm lateral displacements resulted in several diagonal cracks extended further to form diagonal corner to corner cracks and cracks at the lower part of the wall were found to open more widely. Fig. 2.27 illustrates the lateral load versus top displacement relationship of the specimen LW5. By contrast, it can be seen that the crack pattern prior to failure was very similar to that of specimen LW2 but the load- carrying capacity of specimen LW2 was found to be more significant than that of specimen LW5 after maximum base shear was attained.

2.3.5.2 Local Response

Fig. 2.28 plots the strain distribution in the outermost longitudinal bars of specimen LW5. Strains of other longitudinal bars in the wall flange and web along two wall sections, section 1-1 and 2-2, are also presented in Fig. 2.29(a) and 2.29(b), respectively. As shown in Fig. 2.28, the strains are approximately uniform with height above the base level. This can be due to the increasing influence of strut-and-tie action on the wall behavior. The plot of Fig. 2.29(a) shows that the plane section 1-1 does not remain plane after wall drift ratio of 0.67% is attained. As the test progressed, the neutral axis shifted from the wall middle towards the compression face of the wall.

Figs. 2.30(a) and 2.30(b) illustrate the strain distribution in the two horizontal web bars, R bar and T bar, respectively. It can be seen that the strains are concentrated along the main diagonal strut. The plot of Fig. 2.30(b) shows that the strains in the center of the T bar (T14 and T15) are large under both loading directions since this

portion of wall is situated along the main diagonals for both loading directions. The bars are stressed in tension for both loading directions.

2.4 Discussion of Experimental Results

The main experimental results for each specimen tested are listed in Table 2.1. In this section, a simple discussion of experimental results is presented in terms of the observed behavior of each specimen tested. Note that different methods of defining the yield displacements of walls existed. In this testing, the measured displacement at yield state is determined through electrical strain gauges, i.e. when the strain of flexure reinforcements reached the value of 0.2 percent, the displacement and load reading are recorded.

2.4.1 Crack Patterns and Failure Modes

It was observed that the initial flexure cracks located at the lower part of specimens occurred at a drift ratio ranging from 0.1% to 0.17% within the length of the boundary elements. This value of drift ratio may be considered as a serviceability limit of the structural element. With regards to specimens without axial loadings (LW1 and LW4), these initial flexure cracks were developed at a base shear of approximately 204 kN while for specimens subjected to axial loadings the observed base shear corresponding to first cracking of the walls was approximately 270 kN. Further increasing the wall top displacements resulted in all specimens experiencing the yielding of flexure reinforcements in the boundaries at a drift ratio of approximately 0.2%. Moreover, a base force of approximately 300 kN was achieved at the time of flexure yielding for specimens LW1 and LW4 without axial loadings while the observed shear force for corresponding specimens subjected to axial compression was approximately 400 kN. With increasing lateral displacements up to

reaching at the maximum base shear, the flexure-shear cracks of all specimens were extended up to the wall top and a number of diagonal struts were formed to efficiently transfer the lateral loading from the wall top to the bottom. Further cycling the walls to maximum lateral displacements observed no more emergence of new cracks but existing cracks especially those located at the lower part of the walls were found to be widely opened and meanwhile spalling of the concrete cover at the wall base occurred.

In general, all specimens behaved in a flexure manner, characterized by concrete crushing and reinforcement buckling at the wall boundaries. Meanwhile, moderate diagonal cracking of the web and shear sliding at the base for almost all specimens were also observed. It should be noted that the damage of the specimens was mainly located at the lower part of the walls except for wall LW3, whose the damage zone extended upwards and occurred within the left wall flange with a height range from approximately 500 mm to 1000 mm.

2.4.2 Backbone Envelopes of Load-displacement Curves

The backbone envelopes of load-displacement curves have long been recognized to be a critical feature in modeling the inelastic behavior of RC walls. It was generally generated with the curve determined from a monotonic test and herein was constructed by connecting the peaks of recorded lateral load versus top displacement hysteretic loops for the first cycle at each deformation level of the tested specimens. Fig. 2.31 shows the backbone envelopes of load-displacement curves of all specimens tested along with the estimated average flexure capacities (refer to Table 2.1). It can be seen clearly from the figure that the presence of axial loadings significantly increases the strength and the stiffness of the tested walls. Interestingly, for tested walls with or without axial loadings, similar top drift was achieved for

tested walls with only one exception (Specimen LW3). For this specimen, the smallest drift capacity was observed due to the fact that inadequate transverse reinforcements were provided to prevent the longitudinal reinforcements in the boundaries from buckling.

As observed from Table 2.1, all five specimens were capable of developing their flexure strength prior to failure, which is a prerequisite of adequate seismic performance. Moreover, in the case of specimens with construction joints, similar maximum flexure strength was developed by comparing with that of corresponding specimens without construction joints. This observation does clearly indicate that for the specimens tested, sliding shear will not inhibit the development of flexure capacity. Meanwhile, a good agreement between the calculated maximum strength and measured maximum flexure strength was observed which indicates that maximum strengths for all specimens were governed by the maximum flexure strength obtained from inelastic section analysis.

2.4.3 Components of Top Deformation

The top deformation of walls in this testing is mainly caused by flexure displacement, panel shear displacement, and sliding displacement. Note that the flexure component of the total deformation also included the contribution from bond slip in longitudinal bars at the base of the wall. From figures of strain profiles such as Figs. 2.9(a) and 2.9(b) etc. for all five tested specimens, it is found that flexure deflection measured by LVDTs at the left and right side of tested specimens could be overestimated and as such the effect of internal strains should be considered to accurately evaluate the flexure deflections. For this purpose, the plane section of tested specimens is assumed to deflect with respect to the best fit line of internal strain distributions and in this study nonlinear strain distributions along the wall

sections 1-1 and 2-2 for each tested specimen are represented by linear trend lines which consider the internal strains mostly. It is found by comparing the results considering the effects of internal strains that the flexure deformations of tested specimens by use of this kind of LVDT measure arrangement are overestimated by a percentage value ranging from 4.87 to 7.97. Fig. 2.32 illustrates the ratios of three displacement components to the total deformation with respect to wall drift ratios by considering the effect of internal strains on flexure deflection of tested specimens. From the figure, the relative contribution of three displacement components varied in terms of the wall drift ratios with the observed behavior that with increasing lateral displacements up to wall failure the contribution of flexure deformations decreased while the ratios of the other two components tended to rise slightly. In general, the flexure deformations dominated the response because it accounted for more than 50% of the total displacement up to the final loading stage for all specimens.

For the reference wall LW1, the sliding shear components accounted for less than 5% initially in both loading directions, but for 15% close to failure. This varied trend is more pronounced for the specimen LW4 with construction joints at the base due to the fact that the sliding components made up approximately 23% at the final stage and meanwhile, exceeded the contributions from shear displacements which was generally observed to be higher than the sliding contributions in other specimens. However, in the case of walls subjected to axial loading including the Specimen LW5 with construction joints at the base, the contribution of sliding shear did not exceed 13%. This indicated that the axial loading played a favorable role on controlling the wall sliding deformations as expected and under axial compression, the presence of the construction joints at the wall base had a minor effect with respect to sliding, as only up to approximately 13% of the total displacement of Specimen LW5 was due to this mode compared with 10% of that contribution in the

comparable Specimens LW2 and LW3.

In contrast with the contributions of various deformation modes for Specimen LW3 as shown in Fig. 2.32©, the flexure displacement contribution of Specimen LW2 as seen from Fig. 2.32(b) tended to be greater (73% compared with 62% in the positive loading direction) at a drift ratio of 0.67% while the shear components of total displacements becomes lesser at that stage (12% compared with 21% of the total displacements). This suggested that for Specimen LW2 with more content of the transverse reinforcements in wall boundaries, the flexure contribution of the total deformation became greater while the sliding component of both walls was observed to be almost the same. Hence, it can be considered that the content of transverse reinforcement for flanged specimens could have an important effect in achieving a more ductile hysteretic response.

2.4.4 Curvature Distribution along the Wall Height

The wall curvature distribution along the wall height, but only the first cycle at certain drift level is shown in Fig. 2.33 for all specimens tested. It is observed that the curvature was highly concentrated at the bottom region and, for wall region higher than 500 mm from the wall base, the curvature remains constant at a lower level. In negative and positive loading directions, the bottom curvatures of the specimen were observed to be close with respect to certain drift ratios. After attaining the wall drift ratio of approximately 0.33%, the bottom curvatures increased significantly. Moreover, at a drift ratio of 1.0% the average curvature of Specimen LW1 as shown in Fig. 2.33(a) was observed to be approximately 20% higher than that of Specimen LW2 as shown in Fig. 2.33(b). Similar conclusion can be made by comparing the bottom curvatures of Specimen LW4 as shown in Fig. 2.33(d) with that of Specimen LW5 as shown in Fig. 2.33(e). This indicates that higher level of wall average curvatures can be attained for the specimen without

axial loadings (Specimen LW4) than that corresponding to specimens subjected to axial compression (Specimen LW5). Moreover, the rate of increasing wall curvatures tends to rise with respect to the increasing drift ratios of the walls.

2.4.5 Stiffness Characteristics

Previous research [A1, P4, T2] indicated that the true stiffness of the wall elements is significantly lower than that corresponding to gross section properties, even at the serviceability limit state. It is, therefore, essential to evaluate realistic stiffness properties of wall elements which can lead to more accurate modeling and analysis of RC buildings with structural walls. The values of initial stiffness for each specimen in both loading directions, which defined at the first yielding of longitudinal reinforcements at wall boundaries, are listed in Table 2.1. It can be seen from the table that the average values of initial stiffness for the selected specimens were attained to be approximately 78.4 kN/mm for walls subjected to axial compressions, and 60.0 kN/mm for the specimens without axial loadings. Fig. 2.34 demonstrates the detailed stiffness properties of the walls which were evaluated using secant stiffness at the peak of first cycle at each deformation amplitude. As expected, all specimens experienced considerable reduction in stiffness with increasing wall deformations. The secant stiffness of each specimen rapidly dropped to about 55% of its uncracked stiffness by a drift ratio of approximately 0.2% corresponding to the yielding state of the wall. With the increase of the wall top drift, the stiffness of each specimen further decreased and at the final stage in testing which only accounted for 15% of its uncracked stiffness.

By comparing with two other specimens subjected to axial loadings, more severe degradation of stiffness for Specimen LW3 with inadequate transverse reinforcements provided at wall boundaries was observed at a drift ratio of 0.67%.

This indicated that such type of wall suffered more apparent strength degradation when the same displacement amplitude was reached. The degradation ratio of secant stiffness, which evaluated by dividing the values of secant stiffness at the initial loading stages by those corresponding to the final loading stages, was achieved to be about 85% for all specimens with or without axial loadings. This suggested that such low level of axial loadings had a minor effect on the degradation rate of secant stiffness despite the fact that the presence of axial compressions in specimens can lead to higher secant stiffness in contrast with those without axial loadings subjected to at the same drift ratios.

2.4.6 Energy Dissipation

The energy dissipation capacity for each specimen which is calculated from the inner area of load-displacement curves has long been recognized to be of paramount importance in the evaluation of the seismic performance of RC walls. Fig. 2.35 plots the energy dissipation capacity of each specimen with respect to its drift ratios. The observations indicate that prior to yield, rather small amount of energy was dissipated, and thereafter the increase rate of energy dissipation for all specimens tended to rise with increasing top drift ratios. With respect to specimens subjected to axial loadings, Specimens LW2 and LW3 as well as LW5, the amount of energy dissipated was larger than that corresponding to specimens without axial compression. This can be due to the favorable effect of the axial compression with regard to controlling the pinching of hysteretic loops. Moreover, for Specimens LW4 and LW5 with construction joints at the wall base, as expected, lower amount of energy was dissipated in contrast to that dissipated by Specimens LW1 and LW2, respectively which was especially obvious when the wall drift ratio was higher than 0.5%. This can be explained by the presence of construction joints at the wall base which presented excessive sliding shear displacements and therefore led to

significant pinching of hysteretic loops of specimens at the last stages in testing. Meanwhile the amount of energy dissipated by Specimen LW2, which had more dense transverse reinforcement at wall boundaries than corresponding Specimen LW3, was observed slightly larger than that by Specimen LW3 up to a wall drift ratio of 0.5%. But thereafter the increasing rate of energy dissipation for Specimen LW3 was more rapid than that corresponding to Specimen LW2 up to the drift ratio of 0.67%.

Meanwhile, for the purpose of comparing the amount of energy dissipated by individual components for each specimen, the flexure deformations, the shear deformations, and the sliding deformations of all specimens are separated from their top displacements and are plotted against the lateral loadings; Fig. 2.36 only shows the loops of Specimens LW2 and LW4. From the figure, it is evident that the energy dissipated by the flexure deformation is much higher than that from shear or sliding deformations. However for Specimen LW4, the sliding shear displacements contribute more to the total displacement and may significantly influence the flexure behavior of the structure and thus decrease the energy dissipation capacity.

2.5 Extrapolation of Experimental Results

The shear force transfer mechanism of low-rise structural walls has been investigated by many researchers [P1-3, W1, T1] and was well outlined by Park *et al.* [P1] in which the shear force is transferred to the wall base by a middle strut and a truss in the triangular region beside the strut. This mechanism of shear force transfer in low-rise walls tested can be verified by the observations from previous figures (For example, Figs. 2.10(a) and 2.10(b)) which show strain histories of gauges in web horizontal reinforcement of tested specimens. It can be observed apparently in these figures that the tensile strain of the horizontal bars in gauges along the

diagonal strut is normally larger than that of gauges away from the strut.

As such, a strut-and-tie analytical model shown in Fig. 2.37 is proposed to simulate the behavior of the reference wall LW1. The concrete contribution is provided by a direct strut (dashed line) from the loading point to the base of the wall and is kept constant after the onset of diagonal cracking. The inclination of the struts varies from 28.4 to 50.6 degree which is close to the expected inclination of shear-flexure cracks. The cross section of the concrete struts converging to the base of the wall is approximately equal to $A_c = 1.4 \cdot c \cdot b$, where c is the depth of the compression zone as shown in Fig. 2.38(a) calculated by the bending theory, and b is the wall width. The area of the two outer struts, outer strut 1 and 2 as shown in Fig. 2.38(b), is assumed to be $1/3$ and $1/4$ the area of the inner struts, respectively (Fig. 2.38(a)) since the shear force is mainly transferred by the inner diagonal strut. The eight longitudinal bars in the web plus eight longitudinal bars in the flange are clustered in the vertical member AE in the center of the flange. This consideration can be validated from Figs. 2.9(a) and 2.9(b) since the strains of those bars included were observed to be beyond yield strains at the maximum load. The transverse reinforcement within a distance of 1000 mm, including six web horizontal bars in the lower part of the wall, is concentrated in horizontal member CD. The failure load of the truss is assumed when the longitudinal reinforcement is observed to be yielded during tests.

The concrete contribution for shear in tested specimens can be estimated by the strength at the onset of the diagonal cracks V_{cr} which can be detected by the strain gauge attached to the horizontal reinforcement because significant tensile strain is developed at this stage. When the horizontal reinforcement is sufficient to resist the applied shear, the tensile strain of the reinforcement will be stable at a certain level which is generally less than its yield strain, as shown in Fig. 2.10(a). The strengths detected by the gauges attached to the web horizontal bars of tested specimens are listed in Table 2.2. It can be seen that the detected strengths are close to that

calculated by Eqs. (2.6) and (2.7) and it is within the range of approximately 10%. Provided that the direct strut takes the shear force equal to the diagonal crack strength 252.4 kN listed in Table 2.2, the other member forces can be determined. The member forces at the maximum negative strength are presented in Fig. 2.37. By employing this model to Specimen LW1, it can be observed that the average tensile strain predicted by the assumed strut-and-tie model agrees well with the tensile strain history of web horizontal bars in walls tested, as shown in Fig. 2.39. This can be further verified by employing this model to Specimen LW4 as shown in Figs. 2.40 and 2.41.

For Specimen LW2 subjected to axial loads, the proposed strut-and-tie model is appropriately modified as shown in Fig. 2.42. The six longitudinal bars in the web plus eight longitudinal bars in the flange are clustered in the vertical member AF in the center of the flange. This consideration can be validated from Figs. 2.14(a) and 2.14(b) since the strains of those bars included were observed to be beyond yield strains at the maximum load. The contribution of web horizontal reinforcements is considered to be within a distance of 1125 mm which includes 8 horizontal bars in wall web. The angle of struts AC and DE is 31.3° that is close to the average angles of diagonal cracks in the lower part of the wall. The area of the two outer struts, outer strut 1 and 2 as shown in Fig. 2.38(b), is assumed to be $\frac{1}{4}$ and $\frac{1}{2}$ the area of the inner struts (Fig. 2.38(a)), respectively. By use of this model, the average strain of tie CD is evaluated and depicted in Fig. 2.43 which also shows tested strain history of horizontal reinforcement in the lower part of the wall against the applied shear force. It can be observed from Fig. 2.43 that the assumed strut-and-tie model is a reasonable model for the flow of forces. This can be further verified by employing this model to Specimens LW3 and LW5 with axial compression, as shown in Figs. 2.44 – 2.47. The maximum loads for all specimens calculated by the proposed model are listed in Table 2.2. It can be seen that the analytical maximum loads show good correlation with the tested maximum loads. Thus, the proposed model may provide

insights into the force transfer mechanism of low-rise walls with or without axial loadings.

2.6 Conclusions

The experimental tests conducted on five RC walls with an aspect ratio of 1.125, with low axial compression load and limited transverse reinforcement, subjected to cyclic lateral loading simulating a moderate earthquake, showed that:

1. In general, all tested specimens with limited transverse reinforcement behave in a flexure – shear manner and are capable of developing their flexure capacity prior to failure. Values of drift at initial cracking range from 0.1% to 0.17%. Specimens LW1, LW2, LW4 and LW5 generally exhibit more ductile behavior than expected, even if they have insufficient confinement reinforcement corresponding to 70% and 25% of the NZS 3101 and ACI 318 specified quantity of confining reinforcement, respectively. As shown in Table 1, the displacement ductility factors of the four specimens are more than 3.0 and can generally experience average story drift of at least 1% without significant strength degradation. By contrast, Specimen LW3, containing 30% and 10% of the NZS 3101 and ACI 318 specified quantity of confining reinforcement respectively, shows quite critical seismic performance with respect to the strength and deformation capacities achieved. The displacement ductility of Specimen LW3 is observed to be less than 3.0 which proves to exhibit only limited ductile behavior of the wall as stipulated in NZS 3101 code [N1]. As such, it is evident that the NZS 3101 and ACI-318 requirements for wall boundary element confinement can be relaxed for structural walls to some extent.

2. Comparing the results considering internal strains, the flexure deformations of tested specimens by use of this kind of LVDT measure arrangement are overestimated by a percentage value ranging from 4.87 to 7.97.
3. It is also found in the testing that the level of axial compression loadings has a minor effect on the degradation rate of secant stiffness despite the fact that the presence of axial compressions in specimens can lead to higher secant stiffness in contrast with those without axial loadings. For the wall specimens subjected to axial compression loadings, the amount of energy dissipated is larger than that corresponding to specimens without axial compression due to the favorable effect of the axial compression with regard to controlling the pinching of hysteresis loops. The amount of energy dissipation due to shear components does not change much under the condition of axial loadings on the specimen. With regards to the energy dissipation contributed by sliding components, it is found to have increased slightly due to the presence of construction joints at the wall base, but still remains at a low level up to the final stage in testing which is especially apparent in specimens subjected to axial loadings.
4. Two strut-and-tie analytical models for low-rise structural walls with and without axial load respectively, accounting for different contribution of horizontal and longitudinal web reinforcement, are developed to accurately reflect the force transfer mechanisms of low-rise structural walls under cyclic loadings. The tensile strains in horizontal web bars of low-rise structural walls can be predicted by use of the assumed strut-and-tie model which agrees well with the tested data. This provides evidence that the assumed strut-and-tie models are reasonable models for the flow of forces and contribution of web reinforcements in walls tested.

Table 2.1 - Observed strengths and ductility of specimens tested

No.	(a) ρ_k	(b) ρ_v	(c) ρ_b	(d) ρ_s	(e) ρ_{NZS}	(f) ρ_{ACI}	(g) $\frac{P}{(f_c A_c)}$	(1) V_{cr}	(2) $V_{max(test)}$	(3) $V_{max(cal.)}$	(4) $\frac{V_{test}}{V_{cal.}}$	(5) Δ_y	(6) K_t	(7) $\mu_{\Delta_{max}}$	(8) δ_{max}
	%	%	%	%	%	%		kN	kN	kN		mm	kN/mm		%
LW1	0.50	0.50	1.40	0.88	1.20	3.64	0.0	210.0	361.0	321.0	1.12	5.3	60.6	4.3	1.0
								-204.3	-341.0	-321.0	1.06	-5.5	58.4	-4.1	-1.0
LW2	0.50	0.50	1.40	0.88	1.20	3.64	0.05	275.9	579.9	500.0	1.15	5.9	84.7	4.9	1.3
								-264.0	-562.7	-500.0	1.12	-6.6	75.7	-3.3	-1.0
LW3	0.50	0.50	1.40	0.33	1.20	3.64	0.05	270.4	523.8	500.0	1.04	6.0	73.5	2.4	0.67
								-258.5	-561.0	-500.0	1.12	-6.2	83.3	-2.2	-0.5
LW4	0.50	0.50	1.40	0.88	1.20	3.64	0.0	203.5	365.1	321.0	1.14	5.5	58.4	4.0	1.0
								-198.3	-363.4	-321.0	1.13	-6.0	53.5	-3.7	-1.0
LW5	0.50	0.50	1.40	0.88	1.20	3.64	0.05	310.4	520.0	500.0	1.04	6.5	76.9	3.3	1.0
								-323.2	-541.2	-500.0	1.08	-6.0	83.3	-3.4	-1.0

Note: (a) Horizontal web reinforcement ratio; (b) Vertical web reinforcement ratio; (c) Flexural reinforcement ratio in boundary element; (d) Volumetric ratio of transverse reinforcement in boundary element; (e) Volumetric ratio of transverse reinforcement required by NZS code; (f) Volumetric ratio of transverse reinforcement required by ACI code; (g) Axial load ratio;

(1) Observed shear force at first cracking; (2) Maximum observed strength during the test; (3) Maximum ideal flexural strength; (4) The ratio of maximum observed shear strength to ideal flexural strength; (5) Yield displacement when outer boundary longitudinal reinforcements yield; (6) The initial stiffness for the ith specimen; (7) Maximum displacement ductility level; (8) Maximum top drift ratio achieved.

Table 2.2 – Strengths at onset of diagonal cracks of specimens tested and predicted by analytical models

	LW1	LW2	LW3	LW4	LW5
V_{cr} (kN)	268.8	481.5	387.6	268.8	376.3
	-252.4	-441.0	-414.5	-252.4	-380.9
V_c (kN)	295.3	437.3	437.3	295.3	437.3
	-295.3	-437.3	-437.3	-295.3	-437.3
V_{cr} / V_c	0.91	1.10	0.89	0.91	0.86
	0.86	1.01	0.95	0.86	0.87
V_{truss} (kN)	388.8	574.0	573.4	389.7	573.3
	-388.8	-574.0	-573.4	-389.7	-573.3
V_{max} / V_{truss}	0.93	0.99	0.91	0.94	0.91
	0.88	0.98	0.98	0.93	0.94

Fig. 2.1 – Stress-strain relationship for steel reinforcements

Fig. 2.2 – Details of Specimen LW1

Fig. 2.3 – Experimental set-up

Fig. 2.4 – Applied loading history

Fig. 2.5 – LVDTs support arrangements in all specimens tested

Fig. 2.6 – Crack patterns of Specimen LW1

Fig. 2.7 – Lateral load – top displacement relationship of Specimen LW1

Fig. 2.8 – Strain distribution in outermost longitudinal bars of Specimen LW1

Fig. 2.9(a) – Strain profiles of the vertical bars along section 1-1 of Specimen LW1

Fig. 2.9(b) – Strain profiles of the vertical bars along section 2-2 of Specimen LW1

Fig. 2.10(a) – Strain distribution in the horizontal web bar (R bar) of Specimen LW1
(Note: R6 in Figs. 2.10(a), 2.15(a), 2.20(a), 2.25(a) and 2.30(a) represents gauge location rather than bar type)

Fig. 2.10(b) – Strain distribution in the horizontal web bar (T bar) of Specimen LW1

Fig. 2.11 – Crack patterns of Specimen LW2

Fig. 2.12 – Lateral load – top displacement relationship of Specimen LW2

Fig. 2.13 – Strain distribution in outermost longitudinal bars of Specimen LW2

Fig. 2.14 (a) – Strain profiles of the vertical bars along section 1-1 of Specimen LW2

Fig. 2.14(b) – Strain profiles of the vertical bars along section 2-2 of Specimen LW2

Fig. 2.15(a) – Strain distribution in the horizontal web bar (R bar) of Specimen LW2

Fig. 2.15(b) – Strain distribution in the horizontal web bar (T bar) of Specimen LW2

Fig. 2.16 – Crack patterns of Specimen LW3

Fig. 2.17 – Lateral load – top displacement relationship of Specimen LW3

Fig. 2.18 – Strain distribution in outermost longitudinal bars of Specimen LW3

Fig. 2.19(a) – Strain profiles of the vertical bars along section 1-1 of Specimen LW3

Fig. 2.19(b) – Strain profiles of the vertical bars along section 2-2 of Specimen LW3

Fig. 2.20(a) – Strain distribution in the horizontal web bar (R bar) of Specimen LW3

Fig. 2.20(b) – Strain distribution in the horizontal web bar (T bar) of Specimen LW3

Fig. 2.21 – Crack patterns of Specimen LW4

Fig. 2.22 – Lateral load – top displacement relationship of Specimen LW4

Fig. 2.23 – Strain distribution in outermost longitudinal bars of Specimen LW4

Fig. 2.24(a) – Strain profiles of the vertical bars along section 1-1 of Specimen LW4

Fig. 2.24(b) – Strain profiles of the vertical bars along section 2-2 of Specimen LW4

Fig. 2.25(a) – Strain distribution in the horizontal web bar (R bar) of Specimen LW4

Fig. 2.25(b) – Strain distribution in the horizontal web bar (T bar) of Specimen LW4

Fig. 2.26 – Crack patterns of Specimen LW5

Fig. 2.27 – Lateral load – top displacement relationship of Specimen LW5

Fig. 2.28 – Strain distribution in outermost longitudinal bars of Specimen LW5

Fig. 2.29(a) – Strain profiles of the vertical bars along section 1-1 of Specimen LW5

Fig. 2.29(b) – Strain profiles of the vertical bars along section 2-2 of Specimen LW5

Fig. 2.30(a) – Strain distribution in the horizontal web bar (R bar) of Specimen LW5

Fig. 2.30(b) – Strain distribution in the horizontal web bar (T bar) of Specimen LW5

Fig. 2.31 – Backbone envelopes of load-displacement curves for tested specimens

Fig. 2.32 – Contribution of various deformation modes to total displacement of walls

Fig. 2.33 – Wall curvature distribution of all specimens tested

Fig. 2.34 – Secant stiffness of tested walls with respect to drift ratios

Fig. 2.35 – Energy dissipation capacity of each specimen with respect to the drift ratios

Fig. 2.36 – Flexure, shear and sliding displacements of Specimens LW2 and LW4

Fig. 2.37 – Strut-and-tie model of Specimen LW1

Fig. 2.38(a) – Forces acting at wall base section

Fig. 2.38(b) – Forces acting at base section of strut-and-tie model

Fig. 2.39 – Strain history of gauge #T14 in horizontal bars of Specimen LW1

Fig. 2.40 – Strut-and-tie model of Specimen LW4

Fig. 2.41 – Strain history of gauge #T14 in horizontal bars of Specimen LW4

Fig. 2.42 – Strut-and-tie model of Specimen LW2

Fig. 2.43 – Strain history of gauge #T14 in horizontal bars of Specimen LW2

Fig. 2.44 – Strut-and-tie model of Specimen LW3

Fig. 2.45 – Strain history of gauge #T14 in horizontal bars of Specimen LW3

Fig. 2.46 – Strut-and-tie model of Specimen LW5

Fig. 2.47 – Strain history of gauge #T14 in horizontal bars of Specimen LW5

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NOTATIONS

A_g	Gross area of section
A_{ch}	Cross-sectional area of a structural member measured out-to-out of transverse reinforcement
A_{sh}	Total cross-sectional area of transverse confining reinforcement within spacing s
c_c	Distance of the critical neutral axis from the compression edge of the wall section
f'_c	Cylinder strength of concrete
f_y	Yielding stress of reinforcing steel bar
h_c	Cross-sectional dimension of column core measured center-to-center of confining reinforcement
K_i	The initial stiffness for the i th specimen
L_w	Horizontal length of wall
V_{cr}	Observed shear force at first cracking
$V_{\max(test)}$	Maximum observed strength during the test
$V_{\max(ideal)}$	Maximum ideal flexure strength
μ	Displacement ductility factor
$\mu_{\Delta \max}$	Maximum displacement ductility
Δ_y	Yield displacement of the walls
s	Spacing of transverse reinforcement
ϕ_0	Ratio of moment of resistance at overstrength to moment

ρ_h	Horizontal web reinforcement ratio
ρ_v	Vertical web reinforcement ratio
ρ_b	Flexure reinforcement ratio in boundary element
ρ_s	Volumetric ratio of transverse reinforcement in boundary element
$\delta_{\max}$	Maximum top drift ratio achieved

CHAPTER THREE

SEISMIC PERFORMANCE

OF MEDIUM-RISE STRUCTURAL WALLS

WITH LIMITED TRANSVERSE REINFORCEMENT

Abstract

This study is intended to examine available ductility of medium-rise reinforced concrete (RC) structural walls containing less confining reinforcement than that recommended by the New Zealand Concrete Design Code [N1] and American Concrete Institute [A2]. Three RC structural walls with an aspect ratio of 1.625 were tested subjected to low levels of axial compression loading and cyclic lateral loading which simulated a moderate earthquake to examine the structural performance of medium-rise walls with limited transverse reinforcement. Conclusions are reached concerning the displacement capacity, strength capacity, curvature distribution, the secant stiffness degradation and the energy dissipation characteristics shown by the walls on the seismic behavior with limited transverse reinforcement. The influence of axial loads, transverse reinforcements in the wall boundary elements, and the presence of construction joints at the wall based on the seismic behavior of walls are also reported from this study. Towards the end of the chapter, reasonable strut-and-tie models are developed to aid in understanding the force transfer mechanism and contribution of reinforcement in walls tested.

Keywords: Medium-rise structural walls; Boundary elements; Deformation capacity; Limited transverse reinforcement; Seismic performance; Strut-and-tie model

3.1 Introduction and Background

For the past two decades, significant progress has been achieved in understanding the performance of structural walls with full ductility. However, in some cases (such as in low to moderate seismicity) the ductility demand may not be as high as that required by fully ductile response and thus the design concept may not be the same as that for fully ductile walls. Besides in existing structural walls, many of them may possess inherent excessive strength compared with that corresponding to the fully ductile response. To take full advantage of the residual strength, and design with a simple and economical procedure, the structural wall with limited transverse reinforcement, which is expected to exhibit limited ductile behavior, has attracted increasing awareness in recent years. At the same time, some structural walls may possess weak interface like cracks initiated by shrinkage, creep etc or construction joints which may have a significant effect on the structural behavior of walls. However, until now, rather limited research is conducted on the analytical and experimental investigations related to medium-rise walls with or without weak interface though it is a fact that many structural walls existed with such problems.

The superior performance of RC structural walls in buildings has long been recognized [F1] in seismic regions and much more attention has been given to the behavior of short and slender walls, either isolated or coupled with structural frame. A complete literature review of experimental results of 134 short walls with aspect ratio less than 2.0 was given in the research conducted by Wood [W1]. Among the total tested walls reviewed, more than 90% of them had aspect ratios less than 1.0 and only three specimens tested had aspect ratios larger than 1.5. Moreover, most of the aforementioned studies focused on a single aspect ratio and thus the effect of this crucial parameter on the failure mode could only be estimated by comparing results for similar walls tested in different programs.

Based on literature review, Penelis [P6] concluded that the behavior of properly designed walls with aspect ratios larger than 2.0 is dominated by flexure while that of walls with aspect ratios less than 1.0 is dominated by shear. Aspect ratios around 1.5 typically result in complicated predictable behavior, either flexure or shear, or a mixed mode of failure under seismic loading. Recently, Thomas *et al.* [T1-2] tested eleven wall specimens with rectangular sections, six with shear span ratio of 1.5 and five with 1.0, detailed to the current design provisions of EC8 [C1] and ACI 318 [A2]. The main parameters examined in this experimental program and of particular relevance to current research were the aspect ratio, axial load, and presence of construction joints; however, the effect of the transverse reinforcement in wall boundary elements was not studied. As such, it was observed that for structural walls with limited transverse reinforcement, previous studies do not provide adequate and conclusive information with respect to the behavior of such type of medium-rise walls, particularly in the case where the mixed failure modes may dominate for walls specified by an aspect ratio of 1.625.

The main purpose of the present study is to assess the validity of detailing relaxation from those required by current design provisions for fully ductile response. In this study, a description of an experimental investigation of three flanged structural walls subjected to moderate axial compression, with various quantities of transverse reinforcement and the presence of construction joints is presented. Conclusions are drawn concerning the deformation and strength capacity, the secant stiffness degradation and the energy dissipation characteristics shown by the walls.

The present study for medium-rise structural walls with limited transverse reinforcement aims to compile the information of economically design structures which fall between full ductility and elasticity, that is, structures with strengths

greater than that required by seismic loading for fully ductile behavior, or less important structures which do not warrant detailing for full ductility. Moreover, this comprehensive experimental program can present a better understanding on the behavior of limited ductile structural walls with various quantities of transverse reinforcements at wall boundaries and with the presence of construction joints at the wall base. Finally, the proposed strut-and-tie models can offer insights into the concept of shear transfer and the contribution of reinforcement in reinforced concrete squat walls.

3.2 Experimental Program

The three medium-rise shear walls, referred to as Specimens MW1-MW3, were constructed and tested as isolated cantilever walls with an aspect ratio of 1.625. The experimental program presented herein aimed at investigating the performance of medium-rise reinforced concrete walls with limited transverse reinforcement and as such the effects of reinforcement detailing and construction joints at the wall base on failure mode, strength, stiffness, and energy dissipation capacity of walls were investigated.

3.2.1 Material Properties

Ready mixed concrete with 13 mm maximum aggregate specified by a characteristic strength of 35 MPa was used to cast the specimens. Two types of steel bars, high yield steel bar (T bar) with the nominal yield strength of 460 MPa and mild steel bar (R bar) with the nominal yield strength of 250 MPa, were used in all specimens. Fig. 3.1 displayed the typical stress-strain relationships of the bars. Among all bars, T10, R10, and R6 were used in the walls while T13 and T20 were applied at the top and base beams.

3.2.2 Code Provisions for Confining Reinforcement in Plastic Hinge Regions

Due to the existence of different requirements for the amount of confining reinforcement in walls in the ACI 318 [A2] and NZS 3101 [N1] codes, design equations in both design codes to ensure adequate ductility are described as follows.

3.2.2.1 ACI 318 Code Provisions

The required area of hoop reinforcement is given by the larger of

$$A_{sh} = 0.3 s h_c \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f_c'}{f_{yh}} \quad (3.1)$$

and

$$A_{sh} = 0.09 s h_c \frac{f_c'}{f_{yh}} \quad (3.2)$$

where A_{sh} is the total cross-sectional area of transverse confining reinforcement within spacing s and perpendicular to dimension h_c ; s is the spacing of transverse reinforcement measured along the longitudinal axis of the structural member; h_c is the cross-sectional dimension of column core measured center-to-center of confining reinforcement; A_g is the gross area of section; A_{ch} is the cross-sectional area of a structural member measured out-to-out of transverse reinforcement.

3.2.2.2 NZS 3101:1995 Code Provisions

Where the neutral axis depth in the potential yield regions of a wall, computed for the approximate design forces for the ultimate limit state, exceeds:

$$c_c = (0.3\phi_0 / \mu)L_w \quad (3.3)$$

where c_c is the distance of the critical neutral axis from the compression edge of the wall section at the ultimate limit state; ϕ_0 is the ratio of moment of resistance at overstrength to moment resulting from specified earthquake forces, where both moments refer to the base section of wall; μ is the displacement ductility capacity relied on in the design; L_w is the horizontal length of wall.

The following requirements of the transverse reinforcing steel shall be satisfied in that part of the wall section which is subjected to compression strains due to the design forces.

$$A_{sh} = \left(\frac{\mu}{40} + 0.1 \right) s_h h_c \frac{A_g}{A_c} \frac{f_c'}{f_{yh}} \left(\frac{c}{L_w} - 0.07 \right) \quad (3.4)$$

3.2.3 Details of Test Specimens

For medium-rise structural walls located in areas of low to moderate seismicity, it may be appropriate to require reinforcement ratios which are between the gravity load design and the seismic design requirements. The overall dimensions and reinforcement details of the specimens tested are shown in Fig. 3.2. Each test specimen has a web reinforcement ratio of 0.50 percent and 150 mm wide $\times$ 300 mm deep boundary elements which are reinforced with eight mild steel bars of 10 mm diameter, giving a reinforcing ratio of 1.4 percent (minimum code requirement is 1.0 percent). Reduced confinement in the boundary elements for specimens MW1 and MW3 is provided by 6.0 mm diameter closed stirrups (hoops) spaced at 75 mm corresponding to 70% of the transverse confining reinforcement required by NZS 3101 at a limited displacement ductility of 3.0 and 25% of that required by

ACI 318 as shown in Eq. (3.4) and Eq. (3.1), respectively. While for specimen MW2, the vertical spacing of the hoops is meant to be 200 mm which corresponds to 30% and 10% of the transverse confining reinforcement required by NZS 3101 and ACI 318, respectively for seismic detailing of fully ductile walls. Note that the hoops enclosing the main flexure bars in wall boundary elements were detailed in accordance with the requirements stipulated by ACI 318-02: having a bent around 90-deg with a six-diameter extension that engages the longitudinal reinforcement and projects into the interior of the stirrups.

All three medium-rise shear walls tested had the same cross-sectional dimensions and wall height and were reinforced with identical longitudinal steel. Each medium-rise wall specimen in this test was 2000 mm wide, 3000 mm high and 120 mm thick. This provided the value of aspect ratios for all three specimens with 1.625 which was calculated according to the respective wall height of 3250 mm measured by the vertical distance between the lateral loading point and the wall base.

The moulds for the three specimens were set up by connecting the standard steel sheets on which each surface were oiled to make it smooth just prior to the concrete casting. During casting, the concrete of all specimens was compacted primarily by means of a standard internal vibrator. The specimens were cast monolithically in the vertical direction except that for specimens with construction joints at the base, Specimen MW3, it was kept to stand for three days after the concrete for the base beam had been poured, vibrated, and leveled off. Just before the upper part of concrete was poured, the hardened concrete and the reinforcing bars in the construction joint area were brushed to remove any loose particles. Then the base beam concrete surface was moistened and the fresh concrete was poured to the upper part of the moulds. After seven days, the moulds were removed and the

specimens were allowed to expose to laboratory environment until just before testing. In the end, one day prior to testing the outer surface of every specimen was made extra smooth for drawing crack patterns during testing.

3.2.4 Experimental Set-up and Loading History

The test rig used in this study is shown in Fig. 3.3. It consisted of two main independent systems: an in-plane loading system and an in-plane base beam reaction system. The in-plane loading system comprised one horizontal hydraulic actuator which was fixed to the reaction wall and two vertical actuators connected to the strong floor. The test units were subjected to in-plane, reversed cyclic loading from the horizontal double-acting actuator applied at the level of the top steel transfer beam (transfer beam 2) as shown in Fig. 3.3. The hydraulic actuator with 1000 mm stroke possessed a capacity of 1000 kN in compression and tension. The axial loading was applied through two vertical actuators, each with a compression capacity of 1000 kN and 500 mm stroke, attached to the top beam system, as shown in Fig. 3.3.

The base beam of the specimens was fixed to the laboratory floor with twelve high strength rods that prevent uplifting of the specimens and horizontal sliding of the units along the floor during the application of the horizontal loading. Moreover, every high strength rod was prestressed to efficiently restrain the rotation and sliding of the specimen during the test. Constant axial load which corresponds to 0.05 of the cylinder compressive strength of the wall cross section that is equal to $f'_c A_g$ was adopted in the testing program and kept constant during the entire test by applying load control to the vertical actuator. Moreover, hinged connections at the tips of both the vertical and the horizontal actuator prevent any substantial restraint to the rotation of the top of the wall, thus insuring cantilever behavior. After the

total constant axial force was applied, the horizontal loading would be introduced through the top steel beam (transfer beam 2) of the specimens.

In previous tests of ductile members, the displacement loading, in which the level of displacement increases according to the ductility factor ($\mu_{\Delta} = \Delta/\Delta_y$), controls the subsequent cycles. However, since the present test units were meant to exhibit only limited ductility, it was thought more valuable to displace the units to deflections corresponding to selected values of drift due to the fact that the ductility factor ($\mu_{\Delta} = \Delta/\Delta_y$) depends heavily upon the definition of Δ_y , which is not readily identified. As mentioned in the previous research [M1], a value of $\Delta/h_w = 0.01$ is considered a practical limit on the drift to be realistically expected in low-rise structural wall buildings. Herein the end of the test was reached at a drift ratio of 1.33% or the strength dropped to less than 80% of the recorded maximum loading. In this study, the tested specimen was subjected to two cycles at each displacement level except that only one cycle was applied to the specimen at a drift ratio of 1/2000. Fig. 3.4 demonstrates the detailed loading sequences during the testing.

3.2.5 Instrumentation of Wall Specimens

For measurement of top deflection, flexure deformations, and shear deformations, two types of Linear Variable Differential Transducers (LVDT), with 100 mm travel and 50 mm travel, were introduced as shown in Fig. 3.5. One LVDT numbered as L14 with 100 mm travel was installed at the top of the specimen to monitor the top lateral displacement. A total of ten LVDTs along the two vertical edges of the specimens were arranged to measure the flexure deformation. The panel shear deformations were detected by two LVDTs (L4 and L5) distributed along diagonal directions of the panels. Two inclined LVDTs (L2 and L3) with one end of the steel rods at the base beam were used to measure the sliding deformations of the wall

panels. The sliding of base beam was detected by one LVDT named as L1 with 50 mm travel positioned at the strong floor.

FLA type 5 mm-gauge length strain gauges with 10 m vinyl-insulated lead wires were used to measure the local strains of the selected reinforcing bars. The strain gauges were attached to merely one layer of the reinforcing nets such that only the outer layer for the bars of boundary elements was considered to attach the strain gauges due to the symmetric configurations of the wall units. During testing, the strains of the bars were recorded by an automatic datalogger and a strain gauge was deemed no longer reliable when the strain exceeded 0.02.

3.3 Experimental Results

The global behavior, represented by crack patterns and hysteretic loops, and local response such as longitudinal and transverse bar strain distribution of the tested specimens are presented in the following figures. For those concerning the crack pattern at different drift ratios, the dashed lines in the grid lines, indicating the spacing of the reinforcement, represent the negative cracks opened during negative loading while the continuous lines refer to the positive cracks in the positive loading. The blackish areas as shown in the figures represent the splitting of the concrete.

3.3.1 Experimental Results of Specimen MW1

3.3.1.1 Global Behavior

Fig. 3.6 demonstrates the crack patterns and failure modes of specimen MW1 at drift ratios of 0.25%, 0.5% and 1.0% corresponding to the initial cracking stage, crack development stage and final failure stage respectively. Fig. 3.6(a) shows the

initial flexure cracks at the lower part of the reference wall observed at a lateral displacement of approximately 8.13 mm corresponding to a drift ratio of 0.25%. With increasing drift ratios up to 0.5% as shown in Fig. 3.6(b), horizontal flexure cracks spaced at approximately 250 mm (the web horizontal reinforcement spacing) appeared and extended up to 70% of the wall height and moreover, shear cracks propagated from the wall boundaries toward the opposite side and from the bottom upward. Increasing the lateral displacements up to failure, it was observed that the web cracks propagated more extensively toward the opposite side in the lower part of the wall and the concrete in the left wall boundary element was spalling considerably. This suggested that with the increase in lateral displacements, the contribution of shear nature to the wall behavior became more significant with the formation of diagonal shear cracks across the entire web at the final stage of testing. However, at the attainment of wall failure, the opening of these diagonal shear cracks remained small and the values of web horizontal bar strains across the diagonal shear cracks were recorded to be low as shown in Fig. 3.10. This indicated that although the shear nature contributed more to the wall behavior with respect to the increasing lateral displacements up to failure, the overall performance of specimen MW1 was still dominated by flexure.

The lateral load versus the top displacement relationship for specimen MW1 is shown in Fig. 3.7. The ideal strength of $V_i = 375$ kN is obtained from rational section analysis and exceeded by 13% and 12% for the positive and negative loadings, respectively. In the positive loading direction, the observed base shear at the first cracking occurs at a lateral displacement of 8.13 mm is approximately 301.0 kN corresponding to a nominal shear stress of $0.21\sqrt{f'_c}$, and no yielding of reinforcements at this time. As the test progressed, the specimen attained its theoretical flexure strength as the vertical boundary element bars yielded in tension at the wall base as shown in Fig. 3.7. Also, it can be seen from this figure that with

the increase of wall drift ratio to 1%, the maximum base shear as shown in Table 3.1 was achieved as the longitudinal bars in boundary elements yielded in tension almost throughout the entire height of the wall. At this time, a number of diagonal struts were formed and spalling of the concrete cover at the left wall boundary near the base was observed.

3.3.1.2 Local Response

The plots of strains in the outermost longitudinal bar (#A bar) in the boundary elements are presented in Fig. 3.8 in four different positions along the wall height: 50 mm (#A35), 550 mm (#A37), 1300 mm (#A39), and 2300 mm (#A41) above the wall base. Longitudinal strains under both positive and negative loading directions are demonstrated. In the compression boundary element, strains in the selected bar are significantly less than those in the tension boundaries with respect to same wall drift ratios and the compressive strain in these longitudinal bars is generally observed in the region up to a wall height of 500 mm. In the tensile boundary elements, at a wall drift ratio of approximately 0.30%, the longitudinal bar (#A bar) experiences first yielding and strain of the whole bar almost reaches yield strain after attaining a wall drift ratio of 0.67%. Under the negative loading direction, the strain distribution of other longitudinal bars along three different sections is also illustrated in Figs. 3.9(a), 3.9(b), and 3.9(c) respectively. The three wall sections, section 1-1, section 2-2 and section 3-3, as shown in Figs. 3.9(a), 3.9(b), and 3.9(c), respectively, are located at respective wall heights of 50 mm, 550 mm, and 1300 mm above the wall base. As indicated in these figures, with the increase of wall drift ratios the neutral axis depth is observed to shift from the section middle to around 600 mm for the section 3-3 and 400 mm for the section 1-1 calculated from the left flange. The flexure plane section hypothesis can be applied to three wall sections up to a drift ratio of 0.25%.

The strains along each selected horizontal web bars at each displacement peak are shown in Figs. 3.10(a)-3.10(c). The three selected horizontal web bars, R, T, and W bars as shown in the figures, are located at respective wall heights of 250 mm, 750 mm, and 1500 mm above the wall base. The figures show that as the test progressed, the increase of strains along the selected horizontal web bars (R, T, and W bar) are observed to be concentrated along the main diagonal struts during both loading directions (positive direction: R7, R8, T15, W70; negative direction: R5, R6, T14). However, strains remained small in web bars lying on or off the main diagonals. Moreover, the bars in the center of the wall (T and W bar) were observed to be highly strained since this portion of the wall was situated along the main diagonals for both loading directions.

3.3.2 Experimental Results of Specimen MW2

For the purpose of investigating wall behavior caused by employing different quantities of transverse reinforcements, less volumetric ratio of transverse reinforcements were provided to the boundary elements of specimen MW2 in contrast with those in specimen MW1. Hence, the vertical spacing of transverse reinforcements in wall boundaries was modified from 75 mm, which was applied in specimen MW1, to 200 mm for specimen MW2.

3.3.2.1 Global Behavior

The plots of crack patterns and failure modes of specimen MW2 are shown in Fig. 3.11 and typical hysteretic loops along with the ductility and drift capacity of the wall are demonstrated in Fig. 3.12. Fig. 3.11(a) shows that the initial flexure cracks located at the lower part of wall boundary elements are observed at a drift ratio of 0.17% for a base shear of approximately 300 kN. As test progressed, the outermost

flexure reinforcement in left boundary elements experienced yielding at a drift ratio of 0.25% (8.1 mm) corresponding to a base shear of 340 kN and based on this, a magnitude of yield displacement of 9.0 mm is obtained as shown in Fig. 3.12. Displacement by 16.25 mm, the flexure cracks in the boundaries as shown in Fig. 11(b) become denser and extend up to the top of the wall. For the positive loading direction, the maximum base shear of 410.2 kN is achieved at a lateral displacement of 21.37 mm corresponding to a displacement ductility of 2.4. When the wall lateral displacements are further increased to failure, it is observed that approximately 40 degree inclined struts formed and the strength degraded by approximately 15% at a drift ratio of 1.0%.

3.3.2.2 Local Response

Fig. 3.13 plots the strains in the outermost longitudinal bar (#A bar) in the boundary elements under both negative and positive loading directions. In the compression boundary element, strains in the selected bar are significantly less than those in the tension boundaries with respect to same wall drift ratios and do not approach yield until the wall is displaced at a drift ratio of 0.67%. In the tensile boundary elements, at a wall drift ratio of approximately 0.30%, the longitudinal bar (#A bar) experiences first yielding and strain of the whole bar almost reached yield strain after attaining a wall drift ratio of 0.67% where the maximum wall lateral strength is attained during testing. Figs. 3.9(a), 3.9(b), and 3.9(c) present the strain distribution of other longitudinal bars along three different sections under the negative loading direction. As indicated in these figures, with the increase of wall drift ratios, the neutral axis depth is observed to shift from the section middle to around 600 mm ($0.3L_w$) for the section 3-3 and 200 mm ($0.1L_w$) for the section 1-1 calculated from the left flange. For section 2-2 and 3-3 at wall heights of 550 mm and 1300 mm, respectively the planes remain to be plane till the final stage of the

testing. For wall section 1-1, it is observed that at the end of the testing the plane does remain to be plane.

Figs. 3.15(a)-3.15(b) illustrate the strains along each selected horizontal web bars at each displacement peak. The two selected horizontal web bars, R and T bars as shown in Figs. 3.15(a)-3.15(b), are located at respective wall heights of 250 mm, 750 mm above the wall base. As it was found in Specimen MW1, the strains in the horizontal bars are generally small and less than half yield strain except for T bar as shown in Fig. 3.15(b) along the main diagonals (T14, T15 in positive loading direction and T14 under negative loading direction).

3.3.3 Experimental Results of Specimen MW3

For the purpose of investigating the behavior of walls with existing construction joints at the base, Specimen MW3 was tested to make a comparison with the experimental results of Specimen MW1 which subjected to a same level of axial loading but with no construction joints at the wall base.

3.3.3.1 Global Behavior

The crack patterns and the final failure modes of the tested wall are shown in Fig. 3.16. At the beginning of the testing, almost horizontal cracks in the lower part of the wall initially developed at a displacement of 5.6 mm for a base shear of approximately 291.2 kN. Increasing the displacement to 8.3 mm resulted in the first yielding of the flexure reinforcements in the boundary elements as shown in Fig. 3.18. As the test progressed, Specimen MW3 developed its maximum base shear of 410.3 kN at 31.74 mm top lateral displacement. At this time, a large portion of the cracks was formed and several diagonal struts crossed in the middle of the wall web.

Small interconnected horizontal cracks were observed at the base of the wall during further cycling the wall to the final testing stage. Fig. 3.17 illustrates the lateral load versus the top displacement relationship of Specimen MW3.

3.3.3.2 Local Response

The plots of strains in selected longitudinal bars in the boundary elements are shown in Fig. 3.18. It was shown in the figure that strains of the selected bars (#A bar) in the compression boundary element are negligible and in the tensile boundary elements, at a wall drift ratio of approximately 0.25%, the longitudinal bar (#A bar) experienced first yielding and thereafter yield strain of the bar was concentrated up to a wall height of 500 mm from the wall base. The strains of other longitudinal bars along three different sections under the negative loading direction are also presented in Figs. 3.19(a), 3.19(b), and 3.19(c), respectively. As indicated in these figures, with the increase of wall drift ratios the neutral axis depth is observed to shift from the section middle to around 400 mm ($0.2L_w$) for the section 3-3 and 100 mm ($0.05L_w$) for the section 1-1 calculated from the left flange. For section 2-2 and 3-3 at wall heights of 550 mm and 1300 mm respectively, the planes remain to be plane till the final stage of the testing. For wall section 1-1, the flexure plane section hypothesis can not be applied after the attainment of a wall drift ratio of 0.5%.

The strains along each selected horizontal web bar at each displacement peak are shown in Figs. 3.20(a) and 3.20(b), respectively. From these figures, it can be noticed that as the test progressed, the increase of strains along the selected web bars (R and T bars) is observed to be concentrated along the main diagonals during both loading directions (positive direction: R7, R8, T15; negative direction: R5, R6, T14).

3.4 Discussion of Experimental Results

In this section, a simple discussion of experimental results was presented in terms of the observed behavior of each specimen. The strength and deformation characteristics for each specimen are listed in Table 3.1. Note that different methods of defining the yield displacements of walls existed and the yield displacement herein was determined as the displacement when flexure reinforcements at the wall boundary elements yield.

3.4.1 Crack Patterns and Failure Modes

It is observed that the initial flexure cracks within the length of the boundary elements located at the lower part of specimens occurred at a drift ratio ranging from 0.17% to 0.25% prior to flexure yielding of vertical boundary element bars. As test progressed, all three specimens experienced yielding of flexure reinforcements in the boundaries at a drift ratio varied between 0.25% and 0.33% corresponding to an average base shear of approximately 340.0 kN. With increasing lateral displacements until the base shear reaches maximum, the flexure-shear cracks of all specimens are extended up to the wall top and a number of diagonal struts are formed to efficiently transfer the lateral loading from the wall top to the bottom. The recorded strains in vertical boundary element bars of all three specimens are observed to be not uniformly distributed along the wall height, but significantly changed with the increase of wall height on the tension side. Note that the rate of change in tensile force in the boundary reinforcements implies bond forces acting along the bars and is observed to be common in slender structural walls. This suggests that the lateral force is mainly transferred by the integrity of concrete-to-steel bond rather than diagonal compression struts.

Moreover, the strains along selected horizontal web bars are recorded to be concentrated along the main diagonal struts during both the loading directions, but remain small throughout the test. This gives a further verification of the above-mentioned behavior that the specimens are dominated by flexure rather than shear. However, concrete crushing and reinforcement buckling at the wall boundaries as well as shear sliding at the base for all specimens are also observed, but it does not significantly influence the flexure behavior of the walls. Accordingly, it can be concluded that all specimens eventually failed in a flexure – shear mode.

3.4.2 Backbone Envelopes of Load-displacement Curves

The backbone envelopes of load-displacement curves have long been recognized to be a critical feature in modeling the inelastic behavior of RC walls. Generally, it is generated with the curve determined from a monotonic test and herein is constructed by connecting the peaks of recorded lateral load versus top displacement hysteretic loops for the first cycle at each deformation level of the tested specimens. Fig. 3.21 shows the backbone envelopes of load-displacement curves of all specimens tested along with the estimated average flexure and drift capacities (refer to Table 3.1). From the figure, almost linear elastic behavior prior to flexure yielding is observed for all specimens and thereafter the response curve changes rapidly. Moreover, it can be seen that all three specimens are capable of developing their flexure strength prior to failure, which is a prerequisite of adequate seismic performance. Meanwhile, a good agreement between the calculated maximum strength and measured maximum flexure strength is observed which indicates that the maximum strength for each specimen is governed by the maximum flexure strength obtained from inelastic section analysis.

In the case of Specimen MW3 with construction joints, almost same maximum flexure strength is developed by comparing with that of Specimen MW1 without

construction joints but interestingly, slightly larger drift capacity for the Specimen MW3 is observed. This observation does clearly indicate that for this specimen tested, sliding shear does not inhibit the development of strength and deformation capacities. In the case of Specimen MW2, although same drift capacity is achieved by comparing with that of Specimen MW1, the strength degradation for this specimen is observed to be more severe than that of Specimen MW1 after the attainment of the maximum flexure strength corresponding to a wall drift ratio of 0.67%. This can be due to the use of more confinement reinforcements in the specimen which could have a favorable effect on inhibiting severely strength degradation.

3.4.3 Components of Top Deformation

The top deformation of walls in this testing is mainly caused by three components: flexure displacement, panel shear displacement, and sliding displacement. Note that the flexure component of the total deformation also includes the contribution from bond slip in longitudinal bars at the base of the wall. From figures of strain profiles such as Figs. 3.9(a) and 3.9(b) etc. for all three tested specimens, it is found that flexure deflection measured by LVDTs at the left and right side of tested specimens could be overestimated and as such the effect of internal strains should be considered to accurately evaluate the flexure deflections. For this purpose, the plane section of tested specimens is assumed to deflect with respect to the best fit line of internal strain distributions and in this study nonlinear strain distributions along the wall sections 1-1 and 2-2 for each tested specimen are represented by linear trend lines which consider the internal strains mostly. It is found by comparing the results considering the effects of internal strains that the flexure deformations of tested specimens by use of this kind of LVDT measure arrangement are overestimated by a percentage value ranging from 3.78 to 6.41. Fig. 3.22 illustrates the ratios of three

displacement components to the total deformation with respect to wall drift ratios by considering the effect of internal strains on flexure deflection of tested specimens. It can be seen from the figure that as test progressed, the contributions of the flexure deformations reduce slightly while the shear and sliding deformations increase. However, it is recognized that at the final testing stage for all specimens, the flexure deformation still dominates the response as it accounts for more than 50% of the total displacement. Note that the horizontal sliding displacement at the base of the wall is negligible at all stages of testing for all three specimens tested, as only up to approximately 10% of the total displacement was attained for Specimen MW3 with construction joints at the wall base. This indicates that under axial compression, the presence of the construction joints at the wall base have a minor effect with respect to sliding. Meanwhile, the flexure displacement contribution of Specimen MW1 tends to be greater (65% compared with 58% in the positive loading direction) at ultimate stage of testing than that of Specimen MW2 while the shear components of total displacements for Specimen MW1 becomes slightly less at that stage (26% compared with 30% of the total displacements). This suggests that for Specimen MW1 with more content of the transverse reinforcements in wall boundaries, the flexure contribution of the total deformation becomes greater. It is, therefore, considered that the content of transverse reinforcement for flanged specimens can have an important effect in achieving a more ductile hysteretic response.

3.4.4 Curvature Distribution along the Wall Height

Fig. 3.23 shows the average curvature distribution along the wall height, but only the first cycle at certain drift level is presented there for all three specimens. It can be observed that for all specimens, the rate of increase of the wall curvatures tends to rise with respect to the increasing drift ratios of the walls. For Specimen MW1,

the curvature is highly concentrated at the bottom region and for wall region higher than 500 mm from the wall base, the curvature remains constant at a lower level. In negative and positive loading directions, the bottom curvatures of this specimen were observed to be close with respect to same drift ratios and increased significantly after the attainment of the wall drift ratio of approximately 0.33% which occurred just after the onset of first yielding of this specimen. Similar observed trend of wall curvature variations along the wall height for Specimen MW3 are illustrated in Fig. 3.23. However, at a drift ratio of 1.0%, the average curvature of Specimen MW3 is observed to be approximately 60% higher than that of Specimen MW1. In the case of Specimen MW2, unlike the observed behavior of curvature distribution in Specimens MW1 and MW3, the maximum curvature capacity for this specimen is concentrated at a wall height of approximately 500 mm.

3.4.5 Stiffness Characteristics

Previous research [A1, T2, P5] indicated that the true stiffness of the wall elements was significantly lower than that corresponding to gross section properties, even at the serviceability limit state. It is, therefore, essential to evaluate realistic stiffness properties of wall elements which can lead to more accurate modeling and analysis of RC buildings with structural walls. The values of initial stiffness for each specimen in both loading directions defined at the first yielding of longitudinal reinforcements at wall boundaries are listed in Table 3.1. Fig. 3.24 demonstrates the detailed stiffness properties of the walls which were evaluated using secant stiffness at the peak of first cycle at each deformation amplitude. As expected, all specimens experienced considerable reduction in stiffness as increasing wall deformations. At the early stage of testing, the stiffness of each specimen rapidly dropped to about 25% of its uncracked stiffness by a drift ratio of approximately 0.15%. With the

increase of the wall top drift, the stiffness of each specimen further decreased and at the final stage in testing, it remained rather low levels with approximately 30% of its initial stiffness as shown in Table 3.1. Moreover, the degradation ratio of secant stiffness which evaluated by dividing the values of secant stiffness at the initial loading stages by those corresponding to the final loading stages was achieved to be about 85% for all walls tested. This suggested that the content of transverse reinforcements in wall boundary element and the presence of construction joints at the wall base had negligible effects on the stiffness characteristics of the tested walls under such levels of axial compression.

3.4.6 Energy Dissipation

The energy dissipation capacity for each specimen, which is calculated from the inner area of load-displacement curves, has long been recognized to be of paramount importance in the evaluation of the seismic performance of RC walls. Fig. 3.25 shows the energy dissipation capacity of each specimen with respect to its drift ratios. From this figure, it can be seen that prior to yield, rather small amount of energy was dissipated, and thereafter the increase rate of energy dissipation for all specimens tended to rise with increasing top drift ratios. In the case of Specimen MW1 with more content of transverse reinforcements in the wall boundary elements, the amount of energy dissipated was larger than that corresponding to Specimen MW2. This can be due to the favorable effect of the transverse reinforcements in wall boundaries with regard to its ability of developing more ductile behavior for the wall tested. For Specimen MW3 with construction joints at the wall base, although the contribution of sliding components to the total displacements was observed to be small (less than 10%), lower amount of energy was dissipated in contrast to that dissipated by Specimen MW1 which was observed to be 20% lower than that of Specimen MW1 at ultimate stage of testing. This could

be explained by the presence of construction joints at the wall base which led to significant pinching of hysteretic loops of specimens at the final stages in testing.

The amount of energy dissipated by tested walls corresponding to the two different loading cycles is also presented in Fig. 3.25, in which the dashed lines represent the second cycles of the corresponding specimen. It was observed apparently that the initial cycles dissipated more energy than the second loading cycles for all three walls. Meanwhile, for the purpose of comparing the amount of energy dissipated by individual components for each specimen, the flexure deformations and shear deformations as well as sliding deformations of all specimens are separated from their top displacements and are plotted against the lateral loadings as shown in Fig. 3.26. From the figure, it is evident that the energy dissipated by the flexure deformation is much higher than that by shear or sliding deformations.

3.5 Extrapolation of Test Results

The shear force transfer mechanism of squat structural walls has been investigated by many researchers [P1-3, W1] and was well outlined by Park *et al.* [P1] in which the shear force is transferred to the wall base by a middle strut and a truss in the triangular region beside the strut. Similarly, a strut-and-tie analytical model as shown in Fig. 3.27 is proposed to simulate the behavior of Specimen MW1. The concrete contribution is provided by a direct strut (dashed line) from the loading point to the base of the wall and is kept constant after the onset of diagonal cracking. The cross section of the concrete struts converging to the base of the wall is approximately equal to $A_c = 1.4 \cdot c \cdot b$, where c is the depth of the compression zone as shown in Fig. 3.28(a) calculated by the bending theory, and b is the wall width. The area of the two outer struts, outer strut 1 and 2 as shown in Fig. 3.28(b), is assumed to be 1/3 and 1/4 the area of the inner struts (Fig. 3.28(a)) respectively, since the shear force is mainly transferred by the inner diagonal strut.

The angle of struts FG and DE is 28.4° which is close to the average angles of diagonal cracks in the lower part of the wall. The transverse reinforcement within a distance of 1000 mm, including eight web horizontal bars in the lower part of the wall, is concentrated in horizontal member GF and DE. The eight longitudinal bars in the web plus eight longitudinal bars in the flange are clustered in the vertical member AE in the center of the flange. This consideration can be validated since the strains of those bars at the lower part of the wall were observed to be beyond yield strains at the maximum load as shown in Figs. 3.9(a) and 3.9(b). It can also be observed from Fig. 3.9(c) that in the upper part of the wall, the bars with strains beyond yield are mainly concentrated in wall flanges rather than the bars in wall web. This variation agrees well with the reduced tensile force in tie AD compared with those in tie DF and FH. The failure load of the truss is assumed when the longitudinal reinforcement yield which is observed during tests.

The concrete contribution for shear in tested specimens can be estimated by the strength at the onset of the diagonal cracks V_{cr} which can be detected by the strain gauge attached to the horizontal reinforcement because significant tensile strain is developed at this stage. When the horizontal reinforcement is sufficient to resist the applied shear, the tensile strain of the reinforcement will be stable at a certain level which is generally less than its yield strain, as shown in Fig. 3.29. The strengths detected by the gauges attached to the web horizontal bars of tested specimens are listed in Table 3.2. It can be seen that the detected strengths are close to that calculated by NZS 3101 code within the range of approximately 10%. Provided that the direct strut takes the shear force equal to the diagonal crack strength 293.5 kN listed in Table 3.2, the other member forces can be determined. The member forces at the maximum negative strength are presented in Fig. 3.27.

By use of this model, the average strain of ties EF and CD is evaluated and depicted in respective Fig. 3.29(a) and 3.29(b) which also show tested strain history of horizontal reinforcement against the applied shear force. It can be observed that the

average tensile strain predicted by the assumed strut-and-tie model agrees well with the tensile strain history of web horizontal bars in walls tested. Thus, the proposed model may provide insights into the force transfer mechanism and contribution of web reinforcements of medium-rise walls under low axial loadings. This can be further verified by employing this model to Specimens MW2 and MW3, as shown in Figs. 3.30 – 3.31, respectively. Figs. 3.32 and 3.33 illustrate the strain history of the gauge (T14) along diagonal strut in the web horizontal bar (#T bar) for Specimens MW2 and MW3, respectively. Also, the horizontal bar strains calculated by using the proposed strut-and-tie model are presented in Figs. 3.32 and 3.33. It is observed that the strains predicted by use of the assumed strut-and-tie model in horizontal web bars of structural walls agree well with the tested data.

3.6 Conclusions

Three isolated cantilever reinforced concrete walls with an aspect ratio of 1.625 have been tested under cyclic loading up to failure. Strength and deformation capacity characteristics of all specimens tested were summarized in Table 3.1. On the basis of the experimental results presented herein, it shows that all three specimens generally behaved in a flexure manner and were capable of developing their flexure strength prior to failure, which is a prerequisite of adequate seismic performance. Values of drift at initial cracking range from 0.17% to 0.25%. Ultimately, the tested medium-rise structural walls designed for low to moderate seismic areas can generally develop a drift capacity not less than 1%.

The content of transverse reinforcements at the wall boundaries, a reduction to 30% and 10% that required by NZS 3101 and ACI-318 code corresponding to fully ductile walls, might be considered as an effective measure for confining the concrete in the compression zone in terms of the limited ductile performance of

walls. Moreover, for the specimens with more content of the transverse reinforcements in wall boundaries, the flexure contribution of the total deformation becomes greater. This indicates clearly that seismic performance such as drift, ductility and energy dissipation capacity can be enhanced by increasing the amount of the transverse reinforcement at the boundary elements of a wall. It is concluded that the content of transverse reinforcement in wall boundary element and the presence of construction joints at the wall base have negligible effects on the stiffness characteristics of the tested walls under such level of axial compressions.

By decomposing the total lateral deformation into flexure and shear components as well as sliding components, it can be demonstrated that the bulk of the energy dissipation is due to flexure. The amount of energy dissipation due to shear components does not change much under the condition of axial loadings on the specimen. With regards to the energy dissipation contributed by sliding components, it is found to increase slightly due to the presence of construction joints at the wall base, but still remained at a low level up to the final stage in testing. However, comparing the results considering the effects of internal strains, the flexure deformations of tested specimens by use of this kind of LVDT measure arrangement are overestimated by a percentage value ranging from 3.78 to 6.41.

The reasonable strut-and-tie analytical model for medium-rise structural walls, which accounts for contribution of horizontal and longitudinal web reinforcements, is developed to accurately reflect the force transfer mechanisms of medium-rise structural walls under cyclic loadings. The tensile strains in horizontal web bars of structural walls can be predicted by use of the assumed strut-and-tie model which agrees well with the tested data. This provides evidence that the assumed strut-and-tie model is a reasonable model for the flow of forces and contribution of web reinforcements in walls tested.

Table 3.1 - Observed strengths and ductility of specimens tested

No.	(a) ρ_s	(b) ρ_v	(c) ρ_t	(d) ρ_z	(e) ρ_{NZS}	(f) ρ_{ACI}	(g) $\frac{P}{(f_c A_g)}$	(1) V_{cr}	(2) $V_{max(1st)}$	(3) $V_{max(2nd)}$	(4) $\frac{V_{max}}{V_{max(2nd)}}$	(5) Δ_y	(6) K_t	(7) $\mu_{\Delta max}$	(8) δ_{max}
	%	%	%	%	%	%		kN	kN	kN		mm	kN/mm		%
NW1	0.50	0.50	1.40	0.88	1.20	3.64	0.05	301.0	424.6	375.0	1.13	9.1	41.2	3.5	1.0
								-293.3	-422.1	-375.0	1.12	-9.5	39.5	-3.4	-1.0
NW2	0.50	0.50	1.40	0.33	1.20	3.64	0.05	294.1	410.2	375.0	1.09	9.0	41.7	3.6	1.0
								-289.4	-403.4	-375.0	1.07	-9.1	41.2	-3.5	-1.0
NW3*	0.50	0.50	1.40	0.88	1.20	3.64	0.05	291.2	410.3	375.0	1.09	10.0	37.5	4.1	1.3
								-286.3	-392.4	-375.0	1.04	-9.8	38.3	-4.2	-1.3

Note: (a) Horizontal web reinforcement ratio; (b) Vertical web reinforcement ratio; (c) Flexural reinforcement ratio in boundary element; (d) Volumetric ratio of transverse reinforcement in boundary element; (e) Volumetric ratio of transverse reinforcement required by NZS code; (f) Volumetric ratio of transverse reinforcement required by ACI code; (g) Axial load ratio;

(1) Observed shear force at first cracking; (2) Maximum observed strength during the test; (3) Maximum ideal flexural strength; (4) The ratio of maximum observed shear strength to ideal flexural strength; (5) Yield displacement when outer boundary longitudinal reinforcements yield; (6) The initial stiffness for the ith specimen; (7) Maximum displacement ductility level; (8) Maximum top drift ratio achieved.

* denotes the specimen with construction joints at the wall base.

Table 3.2 - Strengths at onset of diagonal cracks of specimens tested and predicted by analytical models

	MW1	MW2	MW3
V_{cr} (kN)	310.5	291.6	295.3
	-293.5	-285.7	-281.4
V_c (kN)	274.0	274.0	274.0
	-274.0	-274.0	-274.0
V_{cr} / V_c	1.13	1.06	1.08
	1.07	1.04	1.03

Fig. 3.1 - Stress-strain relationship for steel reinforcements

Fig. 3.2 - Details of Specimen MW1

Fig. 3.3 – Experimental set-up

Fig. 3.4 – Applied loading history

Fig. 3.5 - LVDTs support arrangements in specimen walls

Fig. 3.6 – Crack patterns of Specimen MW1

Fig. 3.7 – Lateral load – top displacement relationship of Specimen MW1

Fig. 3.8 – Strain distribution in outermost longitudinal bars of Specimen MW1

Fig. 3.9(a) – Strain profiles of the vertical bars along section 1-1 of Specimen MW1

Fig. 3.9(b) – Strain profiles of the vertical bars along section 2-2 of Specimen MW1

Fig. 3.9(c) – Strain profiles of the vertical bars along section 3-3 of Specimen MW1

Fig. 3.10(a) - Strain distribution in the horizontal web bar (R bar) of Specimen MW1

Fig. 3.10(b) - Strain distribution in the horizontal web bar (T bar) of Specimen MW1

Fig. 3.10(c) - Strain distribution in the horizontal web bar (W bar) of Specimen MW1

Fig. 3.11 – Crack patterns of Specimen MW2

Fig. 3.12 – Lateral load – top displacement relationship of Specimen MW2

Fig. 3.13 – Strain distribution in outermost longitudinal bars of Specimen MW2

Fig. 3.14(a) – Strain profiles of the vertical bars along section 1-1 of Specimen MW2

Fig. 3.14(b) – Strain profiles of the vertical bars along section 2-2 of Specimen MW2

Fig. 3.14(c) – Strain profiles of the vertical bars along section 3-3 of Specimen MW2

Fig. 3.15(a) - Strain distribution in the horizontal web bar (R bar) of Specimen MW2

Fig. 3.15(b) - Strain distribution in the horizontal web bar (T bar) of Specimen MW2

Fig. 3.16 – Crack patterns of Specimen MW3

Fig. 3.17 – Lateral load – top displacement relationship of Specimen MW3

Fig. 3.18 – Strain distribution in outermost longitudinal bars of Specimen MW3

Fig. 3.19(a) – Strain profiles of the vertical bars along section 1-1 of Specimen MW3

Fig. 3.19(b) – Strain profiles of the vertical bars along section 2-2 of Specimen MW3

Fig. 3.19(c) – Strain profiles of the vertical bars along section 3-3 of Specimen MW3

Fig. 3.20(a) - Strain distribution in the horizontal web bar (R bar) of Specimen MW3

Fig. 3.20(b) - Strain distribution in the horizontal web bar (T bar) of Specimen MW3

Fig. 3.21 - Backbone envelopes of load-displacement curves for tested specimens

Fig. 3.22 – Contribution of various deformation modes to total displacement of walls

Fig. 3.23 - Wall curvature distribution of tested walls

Fig. 3.24 – Secant stiffness of tested walls with respect to drift ratios

Fig. 3.25 - Energy dissipation capacity of each specimen with respect to the drift ratios

(a) Flexure displacement of Specimen MW1

(a) Shear displacement of Specimen MW1

(a) Sliding displacement of Specimen MW1

(b) Flexure displacement of Specimen MW2

(b) Shear displacement of Specimen MW2

(b) Sliding displacement of Specimen MW2

(c) Flexure displacement of Specimen MW3

(c) Shear displacement of Specimen MW3

(c) Sliding displacement of Specimen MW3

Fig. 3.26 – Flexure, shear and sliding displacements of Specimens MW1 to MW3

Fig. 3.27 – Strut-and-tie model of Specimen MW1

Fig. 3.28(a) - Forces acting at wall base section

Fig. 3.28(b) - Forces acting at base section of strut-and-tie model

(a) Horizontal gauge #T14

(b) Horizontal gauge #W69

Fig. 3.29 - Strain history of gauges in horizontal bars of Specimen MW1

Fig. 3.30 – Strut-and-tie model of Specimen MW2

Fig. 3.31 – Strut-and-tie model of Specimen MW3

Fig. 3.32 - Strain history of the gauge T14 in selected horizontal bar of Specimen MW2

Fig. 3.33 - Strain history of the gauge T14 in selected horizontal bar of Specimen MW3

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NOTATIONS

A_g	Gross area of section
A_{ch}	Cross-sectional area of a structural member measured out-to-out of transverse reinforcement
A_{sh}	Total cross-sectional area of transverse confining reinforcement within spacing s
c_c	Distance of the critical neutral axis from the compression edge of the wall section
f'_c	Cylinder strength of concrete
f_y	Yielding stress of reinforcing steel bar
h_c	Cross-sectional dimension of column core measured center-to-center of confining reinforcement
K_i	The initial stiffness for the i th specimen
L_w	Horizontal length of wall
V_{cr}	Observed shear force at first cracking
$V_{\max(test)}$	Maximum observed strength during the test
$V_{\max(ideal)}$	Maximum ideal flexure strength
μ	Displacement ductility factor
$\mu_{\Delta \max}$	Maximum displacement ductility
Δ_y	Yield displacement of the walls
s	Spacing of transverse reinforcement
ϕ_0	Ratio of moment of resistance at overstrength to moment

ρ_h	Horizontal web reinforcement ratio
ρ_v	Vertical web reinforcement ratio
ρ_b	Flexure reinforcement ratio in boundary element
ρ_s	Volumetric ratio of transverse reinforcement in boundary element
$\delta_{\max}$	Maximum top drift ratio achieved

CHAPTER FOUR

STIFFNESS CHARACTERISTICS OF STRUCTURAL WALLS WITH LIMITED TRANSVERSE REINFORCEMENT

Abstract

This study intends to investigate the stiffness characteristics of reinforced concrete (RC) squat structural walls with limited transverse reinforcement. An analytical approach, combining the inelastic flexure and shear components of deformation, is proposed to properly evaluate the initial stiffness of RC walls tested. In this approach, the flexure deformation is calculated by use of a standard moment-curvature analysis and the shear deformation is determined by applying the truss mechanism to the RC walls studied. Based on this proposed analytical approach, a comprehensive parametric study including a total of 180 combinations is carried out and a simple expression according to this study is proposed to determine the initial stiffness of RC walls studied as a function of three factors: yield strength of the outermost longitudinal reinforcement, applied axial compression and wall aspect ratios. Finally, the proposed stiffness formulae are validated with experimental results and it is found, by comparison with other stiffness predictions, to be more effective.

Keywords: RC structural walls; Limited transverse reinforcement; Stiffness characteristic; Elastic un-cracked stiffness; Cracked analytical stiffness; Initial stiffness

4.1 Introduction and Background

Stiffness properties of all elements of RC wall structures can affect the estimation of the fundamental period, displacements and distributions of internal force response between walls. The magnitude of the initial stiffness, EI_e depends on the intensity and distribution of stress on a wall cross-section as well as the extent of flexure cracking. Flexure cracking causes reduction in net cross sectional area and moment of inertia, and hence reduction in initial flexure rigidity of the wall section. This leads to the accurate prediction of initial stiffness of RC members becoming increasingly difficult. Thus, rough estimates of the stiffness are employed in the analysis of RC members under lateral loads. In practice, the value of 0.35 and 0.70 the gross moment of inertia for cracked and un-cracked walls, respectively is widely employed in the first-order analysis. However, this simplification may not be appropriate in many practical cases as the recommended moment of inertia for walls is independent of the reinforcement content and axial load level. To refine this over-simplified stiffness consideration in beams and columns, several equations [S1, M1] have been presented to properly estimate the initial stiffness of beams and columns up to date and different parameters such as axial loads, concrete compressive strength and shear deformations are considered in these equations. Meanwhile, future recommendations for frame analysis have also been presented to apply them to current design codes.

In the case of RC walls under lateral loads, Priestley *et al* [P2-4] indicated that the NZS 3101 [N1] design code recommendations for estimating the stiffness of RC walls fail to recognize the influence of two important factors: the member strength and the reinforcement grade. For stiffness evaluation of RC walls, it is found that the curvature does not vary significantly with reinforcement grade or axial load level. However, the

wall strength has well been recognized to depend significantly on the axial load and reinforcement content. Thus the recommendations in NZS 3101 design code [N1] for stiffness estimation are found to be over-simplified and may lead to an inaccurate assessment of the wall stiffness. To obtain a more accurate evaluation of wall stiffness as determined from the curvature at first yield and the flexure strength, several proposals on how stiffness values can be more realistically assessed were presented [P3-4]. However, Fenwick and Bull [F1] found that these proposals also proved to be over-simplified. To further improve the stiffness predictions, Fenwick and Bull [F1] analytically investigated a range of slender rectangular walls with uniformly spaced reinforcement and proposed an expression for predicting the initial wall stiffness as a function of concrete strength, reinforcement grade and axial loads. As indicated from the research [F1], the expression proposed for accurately predicting the wall stiffness is only justified for slender rectangular walls for which the response is dominated by flexure and thus the proposals cannot be applied to RC walls with low aspect ratios as their response may be controlled by shear deformations.

This study strives to establish more consistency and accuracy in predicting initial stiffness of these low-rise structural walls with limited transverse reinforcement. Firstly, a brief review of current ACI 318-02 [A1], NZS 3101-1995 [N1] and FEMA 356 provisions [F2] concerning the initial stiffness, EI_e is presented and the need for modifications to the current code provisions is explained. Secondly, to better understand the initial wall stiffness, this study makes a comparison among several stiffness characteristics: un-cracked elastic stiffness, fully cracked wall stiffness and the initial wall stiffness. Thirdly, this study proposes an analytical approach to properly assess the initial flexure and shear stiffness of squat RC walls with limited transverse reinforcement. In this approach, an extensive parametric study including a total of 180

combinations are carried out and the influences of several important parameters on the wall initial stiffness are pointed out. Finally, based on the parametric study, a simple yet accurate formula combining both shear and flexure deformations is presented to predict the initial stiffness of squat RC walls. The obtained analytical results by use of this simple formula is verified against experimental results and compared with alternative recommendations from several existing codes and standards.

4.2 Previous Research in Evaluating Initial Wall Stiffness

In the following sections, the effective moment of inertia, I_e , is defined as the moment of inertia that a uniform elastically responding wall would have, such that when it is subjected to the lateral force that causes first yield, or a strain of 0.002 in the concrete, it sustains the same deflection. The initial stiffness of walls defined as explained above is utilized in several previous researches and current design codes. They are briefly reviewed in the subsequent sections.

4.2.1 Research Conducted by Fenwick and Bull [F1]

The member initial stiffness can be obtained by the integration of curvatures over the cracked and un-cracked sections along the member. The standard beam theory describes the behavior of idealized cracked and un-cracked sections and can be used to determine the initial stiffness theoretically. However, the real flexure response is complicated by bond slip and tension stiffening. Moreover, the presence of axial compression further stiffens the member by delaying the onset of cracking. After considering these, Fenwick and Bull [F1] conducted a parametric study considering three main parameters: axial loads (N), yield strength of longitudinal reinforcement

(MPa) and concrete compressive strength (MPa). The authors went on to propose the following expression as shown in Eq. (4.1) by relating the effective moment of inertia to the moment of inertia of un-cracked concrete section for cantilever walls subjected to flexure deformations predominantly.

$$I_e = 0.267(1 + 4.4 \frac{P_u}{f_c' A_g})(0.62 + \frac{190}{f_y})(0.76 + 0.005 f_c') I_g \quad (4.1)$$

The authors suggested that the expression could be used for slender walls whose behavior is dominated by flexure. Meanwhile, it was assumed that the longitudinal reinforcement was spread uniformly along the walls with rectangular cross sections. In the downside, the expression could not be applied on walls with low aspect ratios as their behavior is normally dominated by deformation associated with shear, also not for walls with other types of reinforcement arrangements and other shapes of wall cross section.

4.2.2 Research Conducted by Paulay and Priestley [P2]

Paulay and Priestley [P2] proposed the following equation as shown in Eq. (4.2) by relating the equivalent moment of inertia of the wall cross section at first yield in the extreme fiber, I_e (mm⁴) to the moment of inertia of the un-cracked gross concrete section, I_g (mm⁴) to determine the initial stiffness of RC structural walls which respond in a flexure manner.

$$I_e = (\frac{100}{f_y} + \frac{P_u}{f_c' A_g}) I_g \quad (4.2)$$

It can be seen from Eq. (4.2) that two important parameters: yield strength of longitudinal reinforcement, f_y (MPa) and axial load ratio, $P_u/f_c' A_g$ are considered to reflect the initial stiffness of flexure walls. Moreover, to account for shear deformation contributions to the wall stiffness, the authors [P2] presented the following expressions as shown in Eqs. (4.3a) and (4.3b) to evaluate the initial stiffness for structural walls with aspect ratios less than 4.0:

$$I_w = \frac{I_e}{1.2 + F} \quad (4.3a)$$

where

$$F = \frac{30I_e}{h_w^2 b_w l_w} \quad (4.3b)$$

In Eqs. (4.3a) and (4.3b), the shear deformation contribution to initial wall stiffness is reflected by employing three overall dimensions: wall height, h_w , wall thickness, b_w and wall length, l_w .

4.2.3 ACI 318-02 [A1]

ACI 318-02 [A1] recommends the application of the following effective stiffness coefficients for walls in the structure. The initial stiffness, EI_e taken as 0.875 of those in the work of MacGregor and Hage [H1], is proposed to be $0.70E_c I_g$ and $0.35E_c I_g$ for un-cracked and cracked walls, respectively.

4.2.4 NZS 3101: 1995 [N1]

Table 4.1 lists the stiffness coefficients recommended by the New Zealand Concrete Structures Standard (1995) [N1] for the elastic seismic analysis of walls. As shown in Table 4.1, the stiffness coefficients are dependent on the axial load, N and the expected inelastic ductility demand, μ . Under the ultimate limit state, three cases designed for ductility $\mu > 1$, that is $\mu = 1.25, 3.0$ and 6.0 , are all assumed to have passed their yield point. Therefore, the lower bound stiffness values (for example, $0.15I_g$ to $0.45I_g$, $0.30A_g$ to $0.80A_g$ for walls under the axial load ratios, $N/f_c'A_g$ from -0.1 to 0.2) should be used in the analysis. However, under service level state, larger stiffness coefficients can be used for the limited ductile cases, that is $\mu = 1.25$ or 3 since there would be less yielding than compared to walls designed with ductility demands of $\mu = 6$. Note that in the case of $\mu = 1.25$, the walls can be analyzed using their fully un-cracked stiffness under the service level earthquake.

4.2.5 FEMA 356 (FEMA 2000) [F2]

FEMA 356, *Prestandard and Commentary for the Rehabilitation of Buildings* (FEMA 2000) [F2] suggests that RC structural walls respond in three different levels corresponding to their aspect ratios. It recognizes that RC walls with an aspect ratio of less than 1.5 and greater than 3.0 are dominated by shear and flexure, respectively, and the walls with intermediate aspect ratio are controlled by both flexure and shear. In FEMA 356, the use of wall initial stiffness values as listed in Table 4.2 that correspond to the secant value at the yield point of the wall is recommended for linear and nonlinear analysis.

4.3 Stiffness Characteristics

There are two common methods used to define the initial wall stiffness as shown in Fig. 4.1. The structure in the first method is cyclic loaded to ± 75 percent of the nominal strength, with the recorded displacement of the structure being extrapolated to the position at the nominal strength level. This defines the ductility displacements, Δ_{y1} as shown in Fig. 4.1. In the second method, the structure is loaded until either the first yield occurs in the longitudinal reinforcement or the maximum compressive strain of concrete reaches 0.002 at the critical section. The displacement at this point, which can be found by integrating the curvatures over the height of the wall, is then extrapolated to the level of the nominal strength and this determines the ductility displacement, Δ_{y2} . Generally, the two methods present similar values and in this study, the second approach is adopted.

For RC structural walls subjected to a specified loading, the final deflection can be modeled through a secant stiffness determined by calibration to tests or detailed analytical models. However, accurate establishment of the secant stiffness for low-rise structural walls is complicated by the interaction of shear, flexure, and axial loading. Considering the interaction of shear, flexure and axial loading, two analytical stiffness of low-rise structural walls, elastic un-cracked stiffness and cracked analytical stiffness, are presented as follows and compared with experimental pre-cracking stiffness and initial stiffness of tested walls. The RC structural walls tested in this study and reported in previous chapters are used for this purpose.

4.3.1 Elastic Un-cracked Stiffness

The elastic un-cracked stiffness, K_e of a RC wall can be readily obtained in a closed-form solution by employing the principle of virtual work. The total deformation of an un-cracked RC wall is separated into components due to flexure, Δ_f , and shear deformations, Δ_{sh} . In the case of cantilever walls which were tested in this program, the total deformation is given by Eq. (4.4)

$$\Delta_t = \Delta_f + \Delta_{sh} = \frac{VH^3}{3E_c I_g} + k \frac{VH}{G_c A_g} \quad (4.4)$$

Thus the elastic un-cracked lateral stiffness, K_e can be determined by the following equation

$$K_e = \frac{V}{\Delta_t} = \frac{1}{\frac{H^3}{3E_c I_g} + \frac{kH}{G_c A_g}} \quad (4.5)$$

where the shear modulus of the concrete, $G_c = E_c / 2(1 + \nu)$ and coefficient, k is related to the cross section shape and defined herein as the ratio of A_g / A_w .

Table 4.3 presents the elastic un-cracked stiffness, K_e for all specimens tested, calculated according to Eq. (4.5). The experimental pre-cracking stiffness, K_{pc} , also shown in Table 4.3 is obtained at a drift ratio of 0.05%, corresponding to the pre-cracking stage during the testing. As listed in Table 4.3, the value of elastic

un-cracked stiffness is much higher than the experimental pre-cracking stiffness obtained and a ratio between them, K_e/K_{pc} is computed to be approximately 3.0:1.0 for medium-rise RC walls. While, for low-rise RC walls with and without axial loadings, the ratio is observed to be 4.5:1.0 and 3.2:1.0, respectively. Accordingly, it can be concluded that even at early stages of testing, the elastic un-cracked stiffness does not represent the true wall stiffness and as such, linear elastic analysis based on the calculated un-cracked stiffness cannot accurately predict the structural forces. This deviation between the elastic un-cracked stiffness and experimental pre-cracking stiffness is likely to be affected not only by the loading history, but also by the material characteristics, dimensions, and curing conditions.

4.3.2 Analytical Cracked Stiffness

The analytical cracked member stiffness, K_c also shown in Table 4.3, is determined using a well-known nonlinear cyclic section analysis computer program: RESPONSE-2000 [B1], without taking the shear deformation into consideration. The cracked section stiffness, $E_c I_e$ at the first yield of the outermost bars in tension flange is calculated by employing the following expression:

$$E_c I_e = \frac{M_y}{\Phi_y} \quad (4.6)$$

Thus the analytical cracked member stiffness, K_c , for a cantilever wall at the stage of the first yield of outermost bars can be expressed as:

$$K_c = \frac{V}{\Delta_{fl}} = \frac{3E_c I_e}{H^3} \quad (4.7)$$

As shown in Table 4.3, the ratio between the analytical cracked member stiffness, K_c , and experimental pre-cracking stiffness, K_{pc} , is achieved to be approximately 2.5:1.0 and 1.7:1.0 for the low-rise and medium-rise RC structural walls, respectively. It indicates that the analytical cracked member stiffness, K_c , is much closer to experimental pre-cracking stiffness, K_{pc} , than the elastic un-cracked stiffness, K_e . Moreover, the value of the analytical cracked member stiffness, K_c , could be further decreased if shear displacements were included in its analysis, and thus the previous comparison would improve. Therefore, it is advisable to use the member analytical cracked stiffness, K_c , rather than elastic stiffness, K_e , as the wall pre-cracking stiffness for elastic analysis.

The experimental initial stiffness, K_i , of the tested walls which is obtained by the previous mentioned approach is also shown in Table 3.4. The ratio, K_c / K_i , between the analytical cracked stiffness, K_c , and experimental initial stiffness, K_i , is observed to be around 3.0:1.0 and 2.2:1.0 for low-rise and medium-rise RC walls, respectively as shown in Table 4.3. It indicates that without considering the contribution of the shear deformation to initial stiffness of RC walls with low aspect ratios, rather large gap can occur between the analytical cracked member stiffness and experimental initial stiffness. Both stiffnesses are obtained at the first yielding of outermost bars in tension flange. It is, therefore, considered to be necessary to account for the contributions of the deformations from both flexure and shear to initial stiffness of walls with low aspect ratios. In the following analysis, a method, considering both flexure and shear deformation, is proposed to properly evaluate the initial stiffness of squat RC structural

walls.

4.3.3 Initial Stiffness

4.3.3.1 Flexure Deformation Determination

To determine the total flexure deformation at the top of the walls, the cantilever wall is divided into five segments along the height of the wall. The curvatures of the five sections along the height of the wall as listed in the column (5) of Table 4.4 can be obtained from their bending moments by the use of moment curvature analysis for each section, when the base section of the wall attains yield moment, M_y . The flexure deformations at the top of the wall, induced by each wall section, are presented in column (6) of Table 4.4. Thus, the total flexure deformation at the top of the wall, Δ_{yf} , can be obtained by cumulating the flexure deformations caused by every segment.

4.3.3.2 Shear Deformation Determination

The concept of modeling cracked RC members as a truss has been around for many years since it provides a more promising way to treat shear. The truss analogy not only presents a clear concept of how a RC wall resists shear, but also properly manages the interaction of flexure, shear and axial load. In truss analogies, longitudinal reinforcement is represented by the longitudinal chords of a truss while transverse steel is represented by transverse tensile ties. The concrete in compression is considered to be the compression chord members. The longitudinal chords and the transverse ties are internally stabilized by the diagonal struts which model the concrete compressive stress field. For simplicity, the longitudinal chords, transverse tensile ties, and diagonal struts

are assumed to be joined together through rigid nodes.

The contribution of the shear deformation to the total deformation in low-rise structural walls under large shear forces can be significant since diagonal cracks must be expected at the service limit state. Hence, in the case of low-rise walls under service conditions, the designer also needs to be able to assess the order of expected shear deformations. Moreover, it is well-recognized that generally, a greater proportion of the load in low-rise structural walls is to be carried by truss action and hence the deformation induced by truss mechanism should be emphasized in this study. Based on this, Park and Paulay [P1] in their study proposed a method to calculate the shear stiffness of short or deep rectangular beams of unit length by using the model of the analogous truss. The shear stiffness is defined as the magnitude of the shear force that when applied to a beam of unit length, will cause unit shear displacement at one end of the beam relative to the other. The shear deformation of these unit length members under certain shear force can be assessed by using the shear stiffness. Fig. 4.2 presents the shear distortions according to the analogy truss mechanism for structural walls studied.

As shown in Fig. 4.2, the horizontal reinforcements and concrete act as tension and strut members, respectively, while the vertical reinforcements form the left and right chords. The total shear distortion includes two components: elongation of the horizontal reinforcements, Δ_S , and the shortening of the compression strut, Δ_C . As indicated in Fig. 4.2, the shear distortion, Δ_V can be defined by

$$\Delta_V = \Delta_S + \Delta_R = \Delta_S + \Delta_C / \sin \alpha \quad (4.8)$$

where α is the inclination of compression strut.

Assuming that the shear force taken by the wall panel is V_s , the stress of horizontal reinforcement can be expressed as

$$f_s = \frac{V_s \cdot s}{d \cot \alpha \cdot A_h} \quad (4.9)$$

where d is the length of wall panel, s is the distance between horizontal reinforcements, and A_h is the area of horizontal reinforcement spaced at a distance s .

Hence the elongation of the horizontal reinforcement becomes

$$\Delta_s = \frac{f_s}{E_s} \cdot d = \frac{V_s \cdot s}{E_s \cot \alpha \cdot A_h} \quad (4.10)$$

The concrete compression stress is obtained

$$f_{cd} = \frac{V_s}{b_w L_{CS} \sin \alpha} \quad (4.11)$$

where b_w is the depth of wall panel and L_{CS} is effective depth of the compression strut as shown in Fig. 4.3.

Hence the shortening of the concrete strut is

$$\Delta_c = \frac{f_{cd}}{E_c} \cdot h_0 / \cos \alpha = \frac{V_s \cdot h_0}{E_c b_w L_{CS} \sin \alpha \cos \alpha} \quad (4.12)$$

where h_0 is the height of the wall panel. By making the appropriate substitution for web horizontal steel content, $\rho_h = A_h / sb_w$, and modular ratio, $n = E_s / E_c$, the shear distortion in the wall panel can be expressed as

$$\theta_v = \frac{\Delta_v}{h_0} = \frac{\Delta_s + \Delta_R}{h_0} = \frac{V_s}{h_0 E_s b_w} \left(\frac{1}{\cot \alpha \cdot \rho_h} + \frac{h_0 n}{L_{cs} \sin^2 \alpha \cos \alpha} \right) \quad (4.13)$$

when $\theta_v = 1$ and $L_{cs} = d \cdot \cos \alpha$, the shear stiffness of the wall panel can be defined by the following expression:

$$K_v = \frac{\rho_h \sin^2 \alpha \cdot \cos^2 \alpha}{\sin^4 \alpha + n \rho_h} E_s b_w d \quad (4.14)$$

Eq. (4.14) indicates that the unit shear stiffness of the wall panel is mainly dependent on the extent of the crack angles. To accurately estimate the theoretical crack angle, Kim and Mander [K1] derived the following equation by considering the energy minimization on the virtual work done by the shear and flexure components.

$$\alpha = \tan^{-1} \left(\frac{\rho_h n + 1.57 \frac{\rho_h}{\rho_v} \times \frac{A_v}{A_g}}{1 + \rho_h n} \right)^{\frac{1}{4}} \quad (4.15)$$

This equation was first employed by Kim and Mander [K1] to analyze the reinforced concrete beam-columns as a truss consisting of a finite number of differential truss elements. Later, Matthew [M2] applied the proposed equation to both shear and flexure

critical structural walls. It was shown in their research that the theoretical crack angles agreed well with the experimentally observed crack angles reported by previous studies. As such, Eq. (4.15) is also employed in this study to conduct the following validation and parametric studies.

Hence the shear displacement caused by the yield lateral force F_y would be

$$\Delta_{yv} = \frac{F_y}{K_v} \cdot h_0 \quad (4.16)$$

4.3.3.3 Combination of Shear and Flexure Response

After the flexure and shear deformation at the top of wall under yield lateral load are obtained, the initial stiffness of walls can be determined as:

$$K_i = \frac{F_y}{\Delta_{yf} + \Delta_{yv}} \quad (4.17)$$

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

$$I_e = \frac{K_i h_w^3}{3E_c} \quad (4.18)$$

4.3.4 Validation of the Proposed Approach

The proposed approach is validated in the following sections by comparing the

theoretical estimations with experimental observations. The experimental results of the eight tested walls reported in the previous chapters are considered here. The theoretical results of the initial stiffness, $K_{i(a)}$, of the eight specimens, together with the experimental results, $K_{i(e)}$, are listed in Table 4.5. To compare the accuracy of the experimental results with the numerical ones, the error value, γ , of numerical results to the experimental ones as shown in Eq. (4.19) is also provided in Table 4.5.

$$\gamma = \frac{|K_{i(a)} - K_{i(e)}|}{K_{i(e)}} \times 100\% \quad (4.19)$$

Table 4.5 shows that the analytical initial stiffness of eight RC walls tested agrees well with the experimental results, as the average error value is observed to be 8.6%. This indicates that the proposed approach can assess initial stiffness of squat RC structural walls with or without axial loads with a satisfactory level of accuracy.

4.4 Parametric Study for Initial Stiffness of Squat Structural Walls

A parametric study based on the proposed approach is carried out to investigate the effect of various parameters on the initial stiffness of low-rise structural walls with limited transverse reinforcement. The primary parameters investigated are: yield strength of longitudinal reinforcement in wall boundaries, f_y , longitudinal reinforcement content in wall boundaries, ρ_b , the axial load ratio, $N/(f_c A_g)$ and aspect ratios, h_w / l_w . The investigated range of the parameters is listed in Table 4.6. It should be noted that the ranges investigated are typical cases for almost all practical low-rise structural walls in low to moderate earthquake regions. All specimens studied

have three common characteristics, namely, 1) aspect ratios less than 2.0, 2) specimens are isolated walls, and 3) solid with symmetrical wall cross section. The tested wall, Specimen LW1, with an aspect ratio of 1.125 as depicted in chapter two is considered to be the reference wall. The longitudinal and transverse reinforcement content in the wall web is kept at a low level of around 0.48 percent and remains to be the same for all specimens studied. The concrete compressive strength of all other specimens studied is the same as that of Specimen LW1.

In the parametric study, the effect of investigated parameters on the initial stiffness of walls is to be presented by the dimensionless stiffness ratio k defined as follows

$$k = \frac{I_e}{I_g} \quad (4.20)$$

Tables 4.7 - 4.9 present the analytical results of the stiffness ratio with the variation of the investigated parameters for longitudinal reinforcement content in wall boundaries of 1.4%, 2.8% and 4.2%, respectively. In the mean time, influence of different parameters investigated on stiffness ratios is also plotted in Figs. 4.4 – 4.6 and is discussed below.

4.4.1 Influence of Aspect Ratio

As listed in Tables 4.7 - 4.9, the stiffness ratio generally increases with the addition of aspect ratios for walls with same reinforcement detailing and under the same level of axial loads. With the increase of aspect ratios from 1.125 to 1.625 and 1.925, stiffness ratios for walls without axial loads rise by approximately 70% and 108%, respectively.

The rate of increase of stiffness ratios becomes more significant for RC walls under an axial load level of 0.1 as it rises by approximately 85% and 130%, respectively.

This variation of stiffness ratios with the change in aspect ratios can also be observed in Figs. 4.4(a) – 4.4(c) corresponding to three different longitudinal reinforcement ratios in wall boundaries: 1.4%, 2.8% and 4.2%, respectively. As wall aspect ratios rise by around 44% and 70%, an increase in stiffness ratios of approximately 80% and 124%, respectively is obtained for walls, subjected to an axial load ratio of 0.05. However, for walls with different longitudinal reinforcement content in wall boundaries, the variation of the stiffness ratios with the change of aspect ratios remain almost the same, which indicates that the longitudinal reinforcement content in wall boundaries plays a minor role in contributing to the initial stiffness of squat RC walls.

4.4.2 Influence of Axial Load

It is generally recognized that the initial stiffness of RC walls should increase with added axial loads due to the decrease in depth of flexure cracks. Although the yield curvature does not vary significantly for walls with or without axial loads, the presence of axial load can effectively increase its strength and thus lead to larger initial flexure stiffness. Analysis shows, in Tables 4.7 – 4.9, that with the added axial loads the stiffness ratio generally increases and approximately 6% and 10% stiffness ratio rise is observed for walls with an aspect ratio of 1.125 when the level of axial load ratio increases from 0.0 to 0.05 and 0.1, respectively.

Figs. 4.5(a) – 4.5(c) clearly illustrate the increase of stiffness ratio with the added axial loads corresponding to three different levels of longitudinal reinforcement content in wall boundaries: 1.4%, 2.8% and 4.2%, respectively. As shown in Figs. 4.5(a) – 4.5(c),

the influence of axial load on the stiffness ratio can be more significant with increase of aspect ratios. It is observed that stiffness ratios for walls with an aspect ratio of 1.625 rise by approximately 13% and 19% with the change of axial load ratio from 0.0 to 0.05 and 0.1, respectively, while for walls with an aspect ratio of 1.925, approximately 16% and 26% rise of stiffness ratios is obtained for walls with the increase of axial load from 0.0 to 0.05 and 0.1, respectively.

4.4.3 Influence of Longitudinal Reinforcement Content in Wall Boundaries

Tables 4.7 – 4.9 shows that with the addition of longitudinal reinforcement content in wall boundaries from 1.4% to 2.8% and 4.2%, stiffness ratios rise slightly by approximately 2% and 5%, respectively for walls without axial loads. The stiffness ratios increase by approximately 2% and 3%, respectively for walls under axial loads of 0.05 with the increase of longitudinal reinforcement content from 1.4% to 2.8% and 4.2%, while this increase reduces to approximately 1.0% and 3.0%, respectively for walls under the axial load ratio of 0.1.

The influence of longitudinal reinforcement ratios in wall boundaries on stiffness ratios is also presented in Figs. 4.6(a) – 4.6(c) for three different aspect ratios: 1.125, 1.625 and 1.925, respectively. From Fig. 4.6, stiffness ratios for walls without axial loads are observed to rise slightly for walls corresponding to all aspect ratios, while for walls under the axial load ratio of 0.1, the stiffness ratios for walls with all aspect ratios studied almost remain the same. This suggests that the influence of longitudinal reinforcement ratios in wall boundaries on stiffness ratios is not significant. For simplicity, the effect of longitudinal reinforcement content in wall boundaries on initial stiffness of RC structural walls with low aspect ratios can be neglected.

4.4.4 Influence of Yield Tensile Strength of Longitudinal Bars in Wall Boundaries

As presented in Tables 4.7 – 4.9, the stiffness ratios generally decrease with the increase of yield strength of outermost longitudinal bars. With the increase of yield strength from 308 MPa to 458 MPa, stiffness ratios of walls without axial loads decrease by approximately 6% for walls with an aspect ratio of 1.125, while there is approximately 10% and 14% reduction in stiffness ratios is observed for walls with an aspect ratio of 1.625 and 1.925, respectively. The influence of yield strength of longitudinal bars on stiffness ratios becomes insignificant for walls with aspect ratios of 1.125 since just 2.5% and 3% reduction in stiffness ratios is observed for walls with axial load ratios of 0.05 and 0.1, respectively.

4.5 Proposed Equation for Moment of Inertia of Structural Walls

Based on the previous parametric study, Eq. (4.21) which considers three above-mentioned parameters investigated: yield tensile strength of steel bars in wall boundaries, axial loads, and aspect ratios is proposed to properly evaluate the initial stiffness of squat structural walls. For simplicity, the influence of longitudinal reinforcement content in wall boundaries on wall initial stiffness is conservatively disregarded.

$$\frac{I_e}{I_g} = 0.19 \left(\frac{100}{f_y} + \frac{N}{f_c' A_g} \right) \left(0.53 + 0.37 \frac{h_w}{L_w} + 0.31 \frac{h_w^2}{L_w^2} \right) \quad (4.21)$$

Where Effective and gross moment of inertia, I_e and I_g : mm⁴; Axial force, N : N; Reinforcement yield strength, f_y and concrete compressive strength, f_c' : MPa; Height and

Length of the wall panel, h_w and L_w : mm; Gross cross section area, A_g : mm².

The analytical results of stiffness ratios for all studied walls (column 1) along with those calculated by employing Eq. (4.21) and Eq. (4.3) are listed in Table 1 at Appendix A. In Appendix A, Column 3 of Table 1 presents the comparisons between the proposed equation (Eq. (4.21)) and the results from the parametric study (Column 1 of Table 1). It is found that the predictions by Eq. (4.21) is quite reasonable since the mean predicted/analytical stiffness ratio, EI_e / EI_g is obtained to be 1.00 with a standard deviation of 0.17 which is the least values compared with those achieved by Eq. (4.3) and current design provisions. Columns 7, 9 and 11 in Table 1 at Appendix A corresponding to those proposed by ACI 318, NZS 3101, and FEMA 356, respectively show that the mean recommended/analytical EI_e / EI_g ratio is found to be 3.37, 2.87, and 4.81 with a standard deviation of 1.20, 1.05, and 1.71, respectively.

Fig. 4.7 illustrates the comparisons of stiffness ratios proposed by the parametric studies, proposed Eq. (4.21) and previous Eq. (4.3) at different aspect ratios of walls studied. In the mean time, the trend lines of the obtained stiffness ratios are also presented in the figure to indicate the variation of the stiffness ratios with the change of axial loads. As can be seen in Fig. 4.7, the prediction by Eq. (4.21) compares better than results calculated by Eq. (4.3) since the mean predicted/analytical stiffness ratio, as shown in Table 1 at Appendix A, for Eq. (4.3) is 1.63 with a standard deviation of 0.34. However, proposed Eq. (4.21) generally presents higher value of stiffness ratios than that of analytical results for walls with an aspect ratio of 1.125.

4.6 Comparisons of Analytical Stiffness Ratios with Tested Results

Results from RC low-rise structural walls tests as listed in Table 4.10 are compared with analytical results using the proposed model, Eq. (4.21) and other provisions previously reviewed. Experimental stiffness values, EI_e from the tests are back calculated by dividing the displacements at the yield point by the tested yield strength with an elastic model. All tested walls have aspect ratios not larger than two and axial load ratios which range from zero to 0.2, which covers almost all conditions likely to be encountered in practice. Yield strengths of outermost longitudinal bars for all specimens range from 300 MPa to 585 MPa. It is believed that the proposed stiffness model is applicable for all values of yield strengths studied. The longitudinal and transverse reinforcement content in the wall web is limited and remains at a low level for all walls selected.

Table 4.10 and Fig. 4.8 presents the comparison between the experimental and calculated stiffness (EI_e / EI_g) for the proposed model and that presented by other proposals. The tested initial stiffness value is reported at column 1 in Table 4.10. Column 3 in Table 4.10 presents the ratio between the stiffness proposed by Eq. (4.21) and the tested results. As shown in Table 4.10, the initial stiffness predicted by Eq. (4.21) agrees well with that from the tested results as a mean stiffness ratio and standard deviation is obtained to be 0.91 and 0.41, respectively. For comparison, Table 4.10 also lists the initial stiffness calculated by Eq. (4.3) as shown at column 4 which a mean stiffness ratio and standard deviation is observed to be 1.48 and 0.54, respectively.

Fig. 4.8 illustrates the comparisons of stiffness ratios, EI_e / EI_g computed by the currently proposed Eq. (4.21), Eq. (4.3) proposed by Paulay *et al.* and NZS 3101 design codes. Tested-to-calculated values of stiffness ratios in Fig. 4.8 indicate that NZS code significantly over-estimates the initial stiffness of low-rise structural walls. Of the two equations proposed, the currently proposed Eq. (4.21) with a standard deviation of 0.41 appears to be more accurate in initial stiffness evaluation than Eq. (4.3) as a standard deviation of 0.54 was obtained.

4.7 Conclusions

This research focuses primarily on RC flanged structural walls with aspect ratios less than two and with limited transverse reinforcement in the wall web. It is believed that this analysis can be extended to walls with a rectangular cross-section since longitudinal reinforcement content in wall boundaries relating to the boundary area is found to have a minor effect on initial stiffness evaluations.

This study develops an effective method to evaluate the initial stiffness of RC low-rise structural walls with limited transverse reinforcement by incorporating both the flexure and shear deformations. The method uses the results of moment curvature analysis to calculate the flexure deformation, and applies the truss mechanism to evaluate the shear deformation. This analytical method of stiffness evaluation is found to be reasonable when compared with the results from the current test program.

Parametric study shows that three critical parameters: yield strength of outermost longitudinal reinforcement, applied axial load, and wall aspect ratios, influence the initial stiffness of low-rise structural walls most. A simple expression to evaluate the

wall initial stiffness accounting for both flexure and shear deformations is proposed and validated with the tested data based on this parametric study. The results obtained are found to be in good agreement with experimental work.

Table 4.1 – Effective section properties by New Zealand Standard (NZS 1995)

Type of member		Checks at ultimate limit state	Checks at serviceability limit state		
			$\mu = 1.25$	$\mu = 3.00$	$\mu = 6.00$
Walls	$N / f_c' A_g = 0.2$	$0.45 I_g$ $0.80 A_g$	I_g A_g	$0.70 I_g$ $0.90 A_g$	$0.45 I_g$ $0.80 A_g$
	$N / f_c' A_g = 0.0$	$0.25 I_g$ $0.50 A_g$	I_g A_g	$0.50 I_g$ $0.75 A_g$	$0.25 I_g$ $0.50 A_g$
	$N / f_c' A_g = -0.1$	$0.15 I_g$ $0.30 A_g$	I_g A_g	$0.40 I_g$ $0.65 A_g$	$0.15 I_g$ $0.30 A_g$

Table 4.2 – Initial stiffness coefficients for linear analysis of walls in FEMA 356

Type of member	Flexure Rigidity	Shear Rigidity	Axial Rigidity
Walls (uncracked)	$0.8 E_c I_g$	$0.4 E_c A_w$	$E_c A_g$
Walls (cracked)	$0.5 E_c I_g$	$0.4 E_c A_w$	$E_c A_g$

Table 4.3 – Stiffness evaluation of all tested walls

No.	K_e kN/mm	K_{pc} kN/mm	K_c kN/mm	K_i kN/mm	K_e / K_{pc}	K_c / K_{pc}	K_c / K_i
LW1	532	120.2	195.0	60.6	4.4	2.5	3.22
LW2	532	174.0	256.6	84.7	3.1	2.4	3.03
LW3	532	150.4	256.6	73.5	3.5	2.8	3.49
LW4	532	119.3	195.0	58.4	4.5	2.5	3.34
LW5	532	168.2	256.6	76.9	3.2	2.5	3.34
MW1	233	83.12	85.1	41.2	2.8	1.7	2.07
MW2	233	81.41	85.1	41.7	2.9	1.7	2.04
MW3	233	75.76	85.1	37.5	3.1	1.8	2.27

Table 4.4 – Flexure deformation determination

(1) Section (i)	(2) Position (mm)	(3) Segment width(mm)	(4) Moment (kNm)	(5) Curvature (rad/m)	(6) Deformation (mm)
0	0	0	0	0	
1	550	500	180.0	0.05×10^{-3}	0.04
2	1300	500	425.2	0.10×10^{-3}	0.10
3	1550	500	507.2	0.25×10^{-3}	0.19
4	1975	350	649.0	0.75×10^{-3}	0.52
5	2200	100	723.6	0.97×10^{-3}	0.21
Specimens LW1 and LW4: Total flexure deformation =					1.00
0	0	0	0	0	
1	550	500	285	0.08×10^{-3}	0.02
2	1300	500	675	0.20×10^{-3}	0.13
3	1550	500	805	0.32×10^{-3}	0.25
4	1975	350	1026	0.80×10^{-3}	0.55
5	2200	100	1143	1.10×10^{-3}	0.24
Specimens LW2, LW3 and LW5: Total flexure deformation =					1.20
0	0	0	0	0	
1	675	750	243	0.08×10^{-3}	0.04
2	1425	750	512	0.20×10^{-3}	0.21
3	2175	750	782	0.32×10^{-3}	0.52
4	2850	600	1025	0.80×10^{-3}	1.37
5	3200	100	1151	1.10×10^{-3}	0.35
Specimens MW1, MW2 and MW3: Total flexure deformation =					2.50

$$(3)_i = (2)_i / 2250 \times M_y$$

$$(6)_i = (5)_i \times (3)_i \times 2000 \quad (2000 \text{ mm is the wall length})$$

Table 4.5 – Experimental and analytical results for initial stiffness
of the eight specimens tested

Specimens No.	Experiment (kN/mm)	Analysis (kN/mm)	Error value γ (%)
LW1	60.6	64.4	6.3
LW2	84.7	67.6	20.2
LW3	73.5	67.6	8.0
LW4	58.4	64.4	10.3
LW5	76.9	67.6	12.1
MW1	41.2	40.7	1.2
MW2	41.7	40.7	2.4
MW3	37.5	40.7	8.5
Average error value =			8.6

Table 4.6 - Parameters investigated

No.	Name	Description	Range Investigated
1	f_y (MPa)	Yield strength of longitudinal reinforcement in wall boundaries	308, 358, 408, 458
2	ρ_b (%)	Longitudinal reinforcement content in wall boundaries	1.4, 2.8, 4.2
3	$N/(f'_c A_g)$	Axial load ratio	0.00, 0.025, 0.05, 0.075, 0.10
4	h_w / l_w	Aspect ratio	1.125, 1.625, 1.925

Table 4.7 – Stiffness ratio, EI_e / EI_g (%) for walls with $\rho_b = 1.4\%$

Item	h_w / l_w											
$\frac{N}{f_c' A_g}$	1.125				1.625				1.925			
	$f_y \text{ (MPa)}$											
	308	358	408	458	308	358	408	458	308	358	408	458
0.0	6.68	6.51	6.43	6.29	11.46	10.92	10.68	10.32	13.89	13.10	12.75	12.23
0.025	6.90	6.82	6.70	6.53	12.09	11.84	11.47	10.95	14.90	14.50	13.94	13.12
0.05	7.01	6.96	6.93	6.84	12.58	12.31	12.17	11.89	15.69	15.26	15.04	14.61
0.075	7.19	7.15	7.06	7.00	13.04	12.88	12.60	12.43	16.46	16.20	15.74	15.48
0.1	7.28	7.20	7.15	7.07	13.35	13.06	12.91	12.68	16.99	16.49	16.24	15.89

Table 4.8 – Stiffness ratio, EI_e / EI_g (%) for walls with $\rho_b = 2.8\%$

Item	h_w / l_w											
$\frac{N}{f_c' A_g}$	1.125				1.625				1.925			
	$f_y \text{ (MPa)}$											
	308	358	408	458	308	358	408	458	308	358	408	458
0.0	6.82	6.73	6.60	6.42	11.88	11.62	11.19	11.02	14.61	14.22	13.53	13.34
0.025	7.03	6.94	6.85	6.79	12.55	12.23	11.95	11.82	15.68	15.17	14.74	14.55
0.05	7.15	7.07	6.99	6.95	12.90	12.65	12.40	12.28	16.26	15.85	15.45	15.26
0.075	7.25	7.19	7.10	7.04	13.27	13.04	12.75	12.56	16.85	16.48	16.01	15.70
0.1	7.31	7.25	7.18	7.12	13.42	13.23	13.02	12.80	17.13	16.78	16.44	16.08

Table 4.9 – Stiffness ratio, EI_e / EI_g (%) for walls with $\rho_b = 4.2\%$

Item	h_w/l_w											
$\frac{N}{f_c A_g}$	1.125				1.625				1.925			
	f_y (MPa)											
	308	358	408	458	308	358	408	458	308	358	408	458
0.0	6.99	6.94	6.91	6.87	12.42	12.26	12.17	12.05	15.49	15.24	15.10	14.91
0.025	7.15	7.07	7.02	6.97	12.92	12.67	12.50	12.37	16.29	15.90	15.62	15.40
0.05	7.24	7.17	7.12	7.07	13.20	12.98	12.82	12.68	16.74	16.39	16.12	15.91
0.075	7.31	7.24	7.19	7.13	13.45	13.22	13.05	12.86	17.16	16.78	16.52	16.20
0.1	7.35	7.30	7.23	7.19	13.58	13.40	13.20	13.05	17.37	17.08	16.75	16.50

Table 4.10 – Comparison of tested versus predicted stiffness

Units	f_y (MPa)	$\frac{h_w}{l_w}$	$\frac{N}{f'_c A_g}$	EI_e / EI_g				
				Tests [1]	Eq. (4.18) [2]	Ratio [3]=[2]/[1]	Eq. (4.3) [4]	Ratio [5]=[4]/[1]
Specimens tested by Synge <i>et al.</i> [S1]								
Wall 1	300	0.5	0.00	0.047	0.050	1.06	0.097	2.06
Wall 3	315	0.5	0.00	0.024	0.048	1.98	0.065	2.71
Specimens tested by Mestyanek [M1]								
Unit 1.0	520	1.0	0.00	0.055	0.044	0.80	0.107	1.95
Unit 1.5	530	1.5	0.00	0.099	0.064	0.64	0.126	1.27
Unit 2.0	500	2.0	0.00	0.138	0.095	0.69	0.143	1.04
Specimens tested by Lefas <i>et al.</i> [L1]								
SW11	470	1.0	0.00	0.090	0.049	0.54	0.123	1.37
SW12	470	1.0	0.10	0.137	0.072	0.52	0.158	1.15
SW13	470	1.0	0.20	0.124	0.095	0.76	0.185	1.49
SW21	470	2.0	0.00	0.226	0.101	0.45	0.160	0.71
SW22	470	2.0	0.10	0.367	0.149	0.41	0.224	0.61
SW23	470	2.0	0.20	0.368	0.196	0.53	0.283	0.77
Specimens tested by Salonikios <i>et al.</i> [T1]								
LSW1	585	1.0	0.00	0.140	0.039	0.28	0.105	0.75
LSW2	585	1.0	0.00	0.093	0.039	0.42	0.105	1.13
LSW3	585	1.0	0.07	0.124	0.055	0.45	0.133	1.07
MSW1	585	1.5	0.00	0.130	0.058	0.44	0.105	0.81
MSW2	585	1.5	0.00	0.048	0.058	1.20	0.105	2.19
MSW3	585	1.5	0.07	0.147	0.081	0.55	0.133	0.90
Specimens tested by Young <i>et al.</i> [Y1]								
WR-20	449	2.0	0.10	0.149	0.153	1.03	0.230	1.54
WR-10	449	2.0	0.10	0.178	0.153	0.86	0.230	1.29
WR-0	449	2.0	0.10	0.117	0.153	1.31	0.230	1.97
WB	342	2.0	0.10	0.138	0.187	1.35	0.245	1.78
Current tested specimens								
LW1	382	1.125	0.00	0.061	0.082	1.35	0.130	2.13
LW2	382	1.125	0.05	0.082	0.095	1.16	0.142	1.73
LW3	382	1.125	0.05	0.072	0.095	1.32	0.136	1.89
LW4	382	1.125	0.00	0.060	0.082	1.37	0.127	2.12
LW5	382	1.125	0.05	0.073	0.095	1.30	0.140	1.92
MW1	382	1.625	0.05	0.124	0.138	1.12	0.190	1.53
MW2	382	1.625	0.05	0.125	0.138	1.11	0.191	1.53
MW3	382	1.625	0.05	0.113	0.138	1.22	0.186	1.65
Mean=						0.91		1.48
Stdev=						0.41		0.54

Fig. 4.1 – Initial stiffness determination [F1]

Fig. 4.2 – Shear distortion of wall panel using analogous truss [P1]

Fig. 4.3 – Compression strut of wall panel

Fig. 4.4 – Influence of wall aspect ratios on stiffness ratios

(a)

(b)

(c)

Fig. 4.5 – Influence of axial load on wall stiffness ratios

Fig. 4.6 – Influence of longitudinal reinforcement ratios in wall boundaries on wall stiffness ratios

Fig. 4.7 – Comparisons of stiffness ratios proposed by the parametric study and equations

Fig. 4.8 – Comparison of initial stiffness between the analytical results and tested data

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NOTATIONS

A_h	Area of horizontal reinforcement spaced at a distance s
A_g	Gross cross section area
A_v	Net cross section area
b_w	Depth of wall panel
d	Length of wall panel
f_c'	Cylinder strength of concrete
f_y	Yielding strength of reinforcement
F_y	Yield lateral force
h_0	Height of the wall panel
h_w	Height of the cantilever wall
k	Dimensionless stiffness ratio
K_c	Analytical cracked member stiffness
K_e	Elastic un-cracked stiffness
K_i	The wall initial stiffness
$K_{i(a)}$	The theoretical initial stiffness
$K_{i(e)}$	The experimental initial stiffness
K_{pc}	Pre-cracking stiffness
K_v	Shear stiffness
L_{CS}	Effective depth of the compression strut

L_w	Length of the cantilever wall
M_y	Yield moment
N	Axial loads
n	Modular ratio
s	Spacing of transverse reinforcement
V_s	Shear force taken by the wall panel
α	Inclination of compression strut
ρ_b	Longitudinal reinforcement content in wall boundaries
ρ_h	Horizontal steel content
ρ_v	Longitudinal steel content
Δ_c	Shortening of the compression strut
Δ_f	Flexure deformation
Δ_{yf}	Flexure displacement caused by the yield lateral force F_y
Δ_{yv}	Shear displacement caused by the yield lateral force F_y
Δ_s	Elongation of horizontal reinforcements
Δ_{sh}	Shear deformation
Δ_y	Yield displacement of the walls
Δ_v	Shear distortion
Φ_y	Yield curvature
γ	The error value

Appendix A

Table 1 – Influence of various parameters on stiffness ratio, EI_e / EI_g (%) of structural walls with low aspect ratios

No.	Anal- ysis [1]	Eq. (4.18) [2]	Ratio [3] = [2]/[1]	Eq. (4.3) [4]	Ratio [5] = [4]/[1]	ACI 318 [6]	Ratio [7] = [6]/[1]	NZS 3101 [8]	Ratio [9] = [8]/[1]	FEMA 356 [10]	Ratio [11] = [10]/[1]
1	6.68	8.23	1.23	14.67	2.20	35.0	5.24	25.0	3.74	50.0	7.49
2	6.51	7.08	1.09	13.49	2.07	35.0	5.38	25.0	3.84	50.0	7.68
3	6.43	6.21	0.97	12.48	1.94	35.0	5.44	25.0	3.89	50.0	7.78
4	6.29	5.53	0.88	11.61	1.85	35.0	5.56	25.0	3.97	50.0	7.95
5	6.82	8.23	1.21	14.67	2.15	35.0	5.13	25.0	3.67	50.0	7.33
6	6.73	7.08	1.05	13.49	2.00	35.0	5.20	25.0	3.71	50.0	7.43
7	6.6	6.21	0.94	12.48	1.89	35.0	5.30	25.0	3.79	50.0	7.58
8	6.42	5.53	0.86	11.61	1.81	35.0	5.45	25.0	3.89	50.0	7.79
9	6.99	8.23	1.18	14.67	2.10	35.0	5.01	25.0	3.58	50.0	7.15
10	6.94	7.08	1.02	13.49	1.94	35.0	5.04	25.0	3.60	50.0	7.20
11	6.91	6.21	0.90	12.48	1.81	35.0	5.07	25.0	3.62	50.0	7.24
12	6.87	5.53	0.81	11.61	1.69	35.0	5.09	25.0	3.64	50.0	7.28
13	6.9	8.86	1.28	15.27	2.21	35.0	5.07	27.5	3.99	50.0	7.25
14	7.01	9.50	1.35	15.82	2.26	35.0	4.99	30.0	4.28	50.0	7.13
15	7.19	10.13	1.41	16.34	2.27	35.0	4.87	32.5	4.52	50.0	6.95
16	7.28	10.77	1.48	16.82	2.31	35.0	4.81	35.0	4.81	50.0	6.87
17	6.82	7.71	1.13	14.16	2.08	35.0	5.13	27.5	4.03	50.0	7.33
18	6.7	6.85	1.02	13.23	1.97	35.0	5.22	27.5	4.10	50.0	7.46
19	6.53	6.17	0.94	12.42	1.90	35.0	5.36	27.5	4.21	50.0	7.66
20	6.96	8.35	1.20	14.79	2.13	35.0	5.03	30.0	4.31	50.0	7.18
21	6.93	7.48	1.08	13.92	2.01	35.0	5.05	30.0	4.33	50.0	7.22
22	6.84	6.80	0.99	13.17	1.93	35.0	5.12	30.0	4.39	50.0	7.31
23	7.15	8.98	1.26	15.37	2.15	35.0	4.90	32.5	4.55	50.0	6.99
24	7.06	8.11	1.15	14.56	2.06	35.0	4.96	32.5	4.60	50.0	7.08
25	7	7.44	1.06	13.87	1.98	35.0	5.00	32.5	4.64	50.0	7.14
26	7.2	9.62	1.34	15.92	2.21	35.0	4.86	35.0	4.86	50.0	6.94
27	7.15	8.75	1.22	15.16	2.12	35.0	4.90	35.0	4.90	50.0	6.99
28	7.07	8.07	1.14	14.52	2.05	35.0	4.95	35.0	4.95	50.0	7.07
29	7.03	8.86	1.26	15.27	2.17	35.0	4.98	27.5	3.91	50.0	7.11
30	6.94	7.71	1.11	14.16	2.04	35.0	5.04	27.5	3.96	50.0	7.20
31	6.85	6.85	1.00	13.23	1.93	35.0	5.11	27.5	4.01	50.0	7.30

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32	6.79	6.17	0.91	12.42	1.83	35.0	5.15	27.5	4.05	50.0	7.36
33	7.15	9.50	1.33	15.82	2.21	35.0	4.90	30.0	4.20	50.0	6.99
34	7.07	8.35	1.18	14.79	2.09	35.0	4.95	30.0	4.24	50.0	7.07
35	6.99	7.48	1.07	13.92	1.99	35.0	5.01	30.0	4.29	50.0	7.15
36	6.95	6.80	0.98	13.17	1.89	35.0	5.04	30.0	4.32	50.0	7.19
37	7.25	10.13	1.40	16.34	2.25	35.0	4.83	32.5	4.48	50.0	6.90
38	7.19	8.98	1.25	15.37	2.14	35.0	4.87	32.5	4.52	50.0	6.95
39	7.1	8.11	1.14	14.56	2.05	35.0	4.93	32.5	4.58	50.0	7.04
40	7.04	7.44	1.06	13.87	1.97	35.0	4.97	32.5	4.62	50.0	7.10
41	7.31	10.77	1.47	16.82	2.30	35.0	4.79	35.0	4.79	50.0	6.84
42	7.25	9.62	1.33	15.92	2.20	35.0	4.83	35.0	4.83	50.0	6.90
43	7.18	8.75	1.22	15.16	2.11	35.0	4.87	35.0	4.87	50.0	6.96
44	7.12	8.07	1.13	14.52	2.04	35.0	4.92	35.0	4.92	50.0	7.02
45	7.15	8.86	1.24	15.27	2.14	35.0	4.90	27.5	3.85	50.0	6.99
46	7.07	7.71	1.09	14.16	2.00	35.0	4.95	27.5	3.89	50.0	7.07
47	7.02	6.85	0.98	13.23	1.88	35.0	4.99	27.5	3.92	50.0	7.12
48	6.97	6.17	0.89	12.42	1.78	35.0	5.02	27.5	3.95	50.0	7.17
49	7.24	9.50	1.31	15.82	2.19	35.0	4.83	30.0	4.14	50.0	6.91
50	7.17	8.35	1.16	14.79	2.06	35.0	4.88	30.0	4.18	50.0	6.97
51	7.12	7.48	1.05	13.92	1.96	35.0	4.92	30.0	4.21	50.0	7.02
52	7.07	6.80	0.96	13.17	1.86	35.0	4.95	30.0	4.24	50.0	7.07
53	7.31	10.13	1.39	16.34	2.24	35.0	4.79	32.5	4.45	50.0	6.84
54	7.24	8.98	1.24	15.37	2.12	35.0	4.83	32.5	4.49	50.0	6.91
55	7.19	8.11	1.13	14.56	2.03	35.0	4.87	32.5	4.52	50.0	6.95
56	7.13	7.44	1.04	13.87	1.95	35.0	4.91	32.5	4.56	50.0	7.01
57	7.35	10.77	1.46	16.82	2.29	35.0	4.76	35.0	4.76	50.0	6.80
58	7.3	9.62	1.32	15.92	2.18	35.0	4.79	35.0	4.79	50.0	6.85
59	7.23	8.75	1.21	15.16	2.10	35.0	4.84	35.0	4.84	50.0	6.92
60	7.19	8.07	1.12	14.52	2.02	35.0	4.87	35.0	4.87	50.0	6.95
61	11.46	11.99	1.05	19.27	1.68	35.0	3.05	25.0	2.18	50.0	4.36
62	10.92	10.32	0.94	17.27	1.58	35.0	3.21	25.0	2.29	50.0	4.58
63	10.68	9.05	0.85	15.65	1.47	35.0	3.28	25.0	2.34	50.0	4.68
64	10.32	8.06	0.78	14.30	1.39	35.0	3.39	25.0	2.42	50.0	4.84
65	11.88	11.99	1.01	19.27	1.62	35.0	2.95	25.0	2.10	50.0	4.21
66	11.62	10.32	0.89	17.27	1.49	35.0	3.01	25.0	2.15	50.0	4.30
67	11.19	9.05	0.81	15.65	1.40	35.0	3.13	25.0	2.23	50.0	4.47
68	11.02	8.06	0.73	14.30	1.30	35.0	3.18	25.0	2.27	50.0	4.54
69	12.42	11.99	0.97	19.27	1.55	35.0	2.82	25.0	2.01	50.0	4.03

70	12.26	10.32	0.84	17.27	1.41	35.0	2.85	25.0	2.04	50.0	4.08
71	12.17	9.05	0.74	15.65	1.29	35.0	2.88	25.0	2.05	50.0	4.11
72	12.05	8.06	0.67	14.30	1.19	35.0	2.90	25.0	2.07	50.0	4.15
73	12.09	12.91	1.07	20.30	1.68	35.0	2.89	27.5	2.27	50.0	4.14
74	12.58	13.84	1.10	21.29	1.69	35.0	2.78	30.0	2.38	50.0	3.97
75	13.04	14.76	1.13	22.24	1.71	35.0	2.68	32.5	2.49	50.0	3.83
76	13.35	12.36	0.93	23.15	1.73	35.0	2.62	35.0	2.62	50.0	3.75
77	11.84	11.24	0.95	18.39	1.55	35.0	2.96	27.5	2.32	50.0	4.22
78	11.47	9.97	0.87	16.84	1.47	35.0	3.05	27.5	2.40	50.0	4.36
79	10.95	8.99	0.82	15.56	1.42	35.0	3.20	27.5	2.51	50.0	4.57
80	12.31	12.16	0.99	19.46	1.58	35.0	2.84	30.0	2.44	50.0	4.06
81	12.17	10.90	0.90	17.98	1.48	35.0	2.88	30.0	2.47	50.0	4.11
82	11.89	9.91	0.83	16.76	1.41	35.0	2.94	30.0	2.52	50.0	4.21
83	12.88	13.08	1.02	20.49	1.59	35.0	2.72	32.5	2.52	50.0	3.88
84	12.6	11.82	0.94	19.07	1.51	35.0	2.78	32.5	2.58	50.0	3.97
85	12.43	10.83	0.87	17.90	1.44	35.0	2.82	32.5	2.61	50.0	4.02
86	13.06	14.01	1.07	21.47	1.64	35.0	2.68	35.0	2.68	50.0	3.83
87	12.91	12.74	0.99	20.11	1.56	35.0	2.71	35.0	2.71	50.0	3.87
88	12.68	11.76	0.93	19.00	1.50	35.0	2.76	35.0	2.76	50.0	3.94
89	12.55	12.91	1.03	20.30	1.62	35.0	2.79	27.5	2.19	50.0	3.98
90	12.23	11.24	0.92	18.39	1.50	35.0	2.86	27.5	2.25	50.0	4.09
91	11.95	9.97	0.83	16.84	1.41	35.0	2.93	27.5	2.30	50.0	4.18
92	11.82	8.99	0.76	15.56	1.32	35.0	2.96	27.5	2.33	50.0	4.23
93	12.9	13.84	1.07	21.29	1.65	35.0	2.71	30.0	2.33	50.0	3.88
94	12.65	12.16	0.96	19.46	1.54	35.0	2.77	30.0	2.37	50.0	3.95
95	12.4	10.90	0.88	17.98	1.45	35.0	2.82	30.0	2.42	50.0	4.03
96	12.28	9.91	0.81	16.76	1.36	35.0	2.85	30.0	2.44	50.0	4.07
97	13.27	14.76	1.11	22.24	1.68	35.0	2.64	32.5	2.45	50.0	3.77
98	13.04	13.08	1.00	20.49	1.57	35.0	2.68	32.5	2.49	50.0	3.83
99	12.75	11.82	0.93	19.07	1.50	35.0	2.75	32.5	2.55	50.0	3.92
100	12.56	10.83	0.86	17.90	1.43	35.0	2.79	32.5	2.59	50.0	3.98
101	13.43	15.68	1.17	23.15	1.72	35.0	2.61	35.0	2.61	50.0	3.72
102	13.23	14.01	1.06	21.47	1.62	35.0	2.65	35.0	2.65	50.0	3.78
103	13.02	12.74	0.98	20.11	1.54	35.0	2.69	35.0	2.69	50.0	3.84
104	12.8	11.76	0.92	19.00	1.48	35.0	2.73	35.0	2.73	50.0	3.91
105	12.92	12.91	1.00	20.30	1.57	35.0	2.71	27.5	2.13	50.0	3.87
106	12.67	11.24	0.89	18.39	1.45	35.0	2.76	27.5	2.17	50.0	3.95
107	12.5	9.97	0.80	16.84	1.35	35.0	2.80	27.5	2.20	50.0	4.00

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108	12.37	8.99	0.73	15.56	1.26	35.0	2.83	27.5	2.22	50.0	4.04
109	13.2	13.84	1.05	21.29	1.61	35.0	2.65	30.0	2.27	50.0	3.79
110	12.98	12.16	0.94	19.46	1.50	35.0	2.70	30.0	2.31	50.0	3.85
111	12.82	10.90	0.85	17.98	1.40	35.0	2.73	30.0	2.34	50.0	3.90
112	12.68	9.91	0.78	16.76	1.32	35.0	2.76	30.0	2.37	50.0	3.94
113	13.45	14.76	1.10	22.24	1.65	35.0	2.60	32.5	2.42	50.0	3.72
114	13.22	13.08	0.99	20.49	1.55	35.0	2.65	32.5	2.46	50.0	3.78
115	13.05	11.82	0.91	19.07	1.46	35.0	2.68	32.5	2.49	50.0	3.83
116	12.86	10.83	0.84	17.90	1.39	35.0	2.72	32.5	2.53	50.0	3.89
117	13.58	15.68	1.15	23.15	1.70	35.0	2.58	35.0	2.58	50.0	3.68
118	13.4	14.01	1.05	21.47	1.60	35.0	2.61	35.0	2.61	50.0	3.73
119	13.2	12.74	0.97	20.11	1.52	35.0	2.65	35.0	2.65	50.0	3.79
120	13.05	11.76	0.90	19.00	1.46	35.0	2.68	35.0	2.68	50.0	3.83
121	13.89	14.70	1.06	21.00	1.51	35.0	2.52	25.0	1.80	50.0	3.60
122	13.1	12.65	0.97	18.65	1.42	35.0	2.67	25.0	1.91	50.0	3.82
123	12.75	11.10	0.87	16.78	1.32	35.0	2.75	25.0	1.96	50.0	3.92
124	12.23	9.89	0.81	15.24	1.25	35.0	2.86	25.0	2.04	50.0	4.09
125	14.61	14.70	1.01	21.00	1.44	35.0	2.40	25.0	1.71	50.0	3.42
126	14.22	12.65	0.89	18.65	1.31	35.0	2.46	25.0	1.76	50.0	3.52
127	13.53	11.10	0.82	16.78	1.24	35.0	2.59	25.0	1.85	50.0	3.70
128	13.34	9.89	0.74	15.24	1.14	35.0	2.62	25.0	1.87	50.0	3.75
129	15.49	14.70	0.95	21.00	1.36	35.0	2.26	25.0	1.61	50.0	3.23
130	15.24	12.65	0.83	18.65	1.22	35.0	2.30	25.0	1.64	50.0	3.28
131	15.1	11.10	0.74	16.78	1.11	35.0	2.32	25.0	1.66	50.0	3.31
132	14.91	9.89	0.66	15.24	1.02	35.0	2.35	25.0	1.68	50.0	3.35
133	14.9	15.83	1.06	22.24	1.49	35.0	2.35	27.5	1.85	50.0	3.36
134	15.69	16.97	1.08	23.43	1.49	35.0	2.23	30.0	1.91	50.0	3.19
135	16.46	18.10	1.10	24.58	1.49	35.0	2.13	32.5	1.97	50.0	3.04
136	16.99	15.16	0.89	25.70	1.51	35.0	2.06	35.0	2.06	50.0	2.94
137	14.5	13.78	0.95	19.97	1.38	35.0	2.41	27.5	1.90	50.0	3.45
138	13.94	12.23	0.88	18.16	1.30	35.0	2.51	27.5	1.97	50.0	3.59
139	13.12	11.02	0.84	16.68	1.27	35.0	2.67	27.5	2.10	50.0	3.81
140	15.26	14.91	0.98	21.24	1.39	35.0	2.29	30.0	1.97	50.0	3.28
141	15.04	13.36	0.89	19.49	1.30	35.0	2.33	30.0	1.99	50.0	3.32
142	14.61	12.15	0.83	18.06	1.24	35.0	2.40	30.0	2.05	50.0	3.42
143	16.2	16.05	0.99	22.46	1.39	35.0	2.16	32.5	2.01	50.0	3.09
144	15.74	14.50	0.92	20.77	1.32	35.0	2.22	32.5	2.06	50.0	3.18
145	15.48	13.28	0.86	19.40	1.25	35.0	2.26	32.5	2.10	50.0	3.23

146	16.49	17.18	1.04	23.65	1.43	35.0	2.12	35.0	2.12	50.0	3.03
147	16.24	15.63	0.96	22.01	1.36	35.0	2.16	35.0	2.16	50.0	3.08
148	15.89	14.42	0.91	20.68	1.30	35.0	2.20	35.0	2.20	50.0	3.15
149	15.68	15.83	1.01	22.24	1.42	35.0	2.23	27.5	1.75	50.0	3.19
150	15.17	13.78	0.91	19.97	1.32	35.0	2.31	27.5	1.81	50.0	3.30
151	14.74	12.23	0.83	18.16	1.23	35.0	2.37	27.5	1.87	50.0	3.39
152	14.55	11.02	0.76	16.68	1.15	35.0	2.41	27.5	1.89	50.0	3.44
153	16.26	16.97	1.04	23.43	1.44	35.0	2.15	30.0	1.85	50.0	3.08
154	15.85	14.91	0.94	21.24	1.34	35.0	2.21	30.0	1.89	50.0	3.15
155	15.45	13.36	0.86	19.49	1.26	35.0	2.27	30.0	1.94	50.0	3.24
156	15.26	12.15	0.80	18.06	1.18	35.0	2.29	30.0	1.97	50.0	3.28
157	16.85	18.10	1.07	24.58	1.46	35.0	2.08	32.5	1.93	50.0	2.97
158	16.48	16.05	0.97	22.46	1.36	35.0	2.12	32.5	1.97	50.0	3.03
159	16.01	14.50	0.91	20.77	1.30	35.0	2.19	32.5	2.03	50.0	3.12
160	15.7	13.28	0.85	19.40	1.24	35.0	2.23	32.5	2.07	50.0	3.18
161	17.13	19.23	1.12	25.70	1.50	35.0	2.04	35.0	2.04	50.0	2.92
162	16.78	17.18	1.02	23.65	1.41	35.0	2.09	35.0	2.09	50.0	2.98
163	16.44	15.63	0.95	22.01	1.34	35.0	2.13	35.0	2.13	50.0	3.04
164	16.08	14.42	0.90	20.68	1.29	35.0	2.18	35.0	2.18	50.0	3.11
165	16.29	15.83	0.97	22.24	1.37	35.0	2.15	27.5	1.69	50.0	3.07
166	15.9	13.78	0.87	19.97	1.26	35.0	2.20	27.5	1.73	50.0	3.14
167	15.62	12.23	0.78	18.16	1.16	35.0	2.24	27.5	1.76	50.0	3.20
168	15.4	11.02	0.72	16.68	1.08	35.0	2.27	27.5	1.79	50.0	3.25
169	16.74	16.97	1.01	23.43	1.40	35.0	2.09	30.0	1.79	50.0	2.99
170	16.39	14.91	0.91	21.24	1.30	35.0	2.14	30.0	1.83	50.0	3.05
171	16.12	13.36	0.83	19.49	1.21	35.0	2.17	30.0	1.86	50.0	3.10
172	15.91	12.15	0.76	18.06	1.14	35.0	2.20	30.0	1.89	50.0	3.14
173	17.16	18.10	1.05	24.58	1.43	35.0	2.04	32.5	1.89	50.0	2.91
174	16.78	16.05	0.96	22.46	1.34	35.0	2.09	32.5	1.94	50.0	2.98
175	16.52	14.50	0.88	20.77	1.26	35.0	2.12	32.5	1.97	50.0	3.03
176	16.2	13.28	0.82	19.40	1.20	35.0	2.16	32.5	2.01	50.0	3.09
177	17.37	19.23	1.11	25.70	1.48	35.0	2.01	35.0	2.01	50.0	2.88
178	17.08	17.18	1.01	23.65	1.38	35.0	2.05	35.0	2.05	50.0	2.93
179	16.75	15.63	0.93	22.01	1.31	35.0	2.09	35.0	2.09	50.0	2.99
180	16.5	14.42	0.87	20.68	1.25	35.0	2.12	35.0	2.12	50.0	3.03
Mean =			1.00		1.63		3.37		2.87		4.81
Stdev =			0.17		0.34		1.20		1.05		1.74

Note: Ratio means ratio of proposed results to analytical ones

CHAPTER FIVE

FINITE ELEMENT PARAMETRIC STUDY OF THE BEHAVIOR OF STRUCTURAL WALLS WITH LIMITED TRANSVERSE REINFORCEMENT

Abstract

The main objective of this study is to provide an analytical approach to better understand the global responses of eight RC structural walls tested and described in the companion study. For this purpose, a nonlinear finite element analytical procedure incorporating microscopic material models for these RC walls is used in this study. The simulated global responses such as strength capacity, stiffness characteristics and energy dissipation capacity of studied RC walls under reversed seismic loadings are found to correlate well with the experimental ones by employing this analytical procedure. A comprehensive parametric study is carried out to report the influence of several paramount parameters: axial loads, longitudinal reinforcements in the wall boundary elements, aspect ratio, area ratio of boundary columns and the presence of construction joints at the wall base on the global behavior of RC walls. Conclusions are drawn concerning the effects of these parameters on global responses in terms of the strength capacity, secant stiffness characteristic, energy dissipation capacity and equivalent damping of the RC walls studied.

Keywords: RC structural walls; Nonlinear finite element; Global response; Energy dissipation capacity; Equivalent damping; Parametric study

5.1 Introduction and Background

Singapore and Peninsular Malaysia are in a seismic risk area with an active earthquake belt comprising the Sumatra Fault and the subduction zone at about 350 km away from the closest point. Although there has never been any earthquake damage to Singapore and Peninsular Malaysia, ground tremors have been felt in these areas many times, and the incidents have increased significantly in number over the last three decades. Strong tremors were felt in buildings of Singapore due to the recent north Sumatra Earthquake. However, in these low to moderate seismic regions like Singapore and Malaysia, RC structural walls are normally designed in accordance with the British Standard: BS 8110 which excludes the influence of seismic loading. Hence, these RC structural walls are usually non-seismically or limited seismically detailed which would imply a lack of confinement reinforcement in wall boundaries. In such non-seismically or limited seismically detailed structural walls, extensive damages may occur under the earthquake excitation as a result of excessive shear deformation and severe strength degradation. Therefore, it is of great concern that the structural performance of these walls may not be adequate to sustain earthquake-induced loads in regions of low to moderate seismicity like Singapore and Malaysia. As such, experimental and numerical studies are needed to investigate the seismic performance of non-seismically detailed structural walls in terms of their strength and deformation characteristics as well as energy dissipation capacity.

Up to date, many experimental and analytical studies have been performed [D2, E1, F1, K1-K6, P4-5, T1-2, W2] to predict the nonlinear behavior of isolated structural walls subjected to load reversals. Although reliable information on the behavior of RC structural walls can be obtained through experimental studies as presented in the

companion studies, it is a time consuming process. As such, in the past decades, numerous analytical models were developed in the modeling of RC structural walls. Most of them developed were macroscopic models [V1, L1, K4-5, O1] for RC structural walls due to their easy application and in these studies great success has been achieved at the element level [S1, X1]. However, these analytical results are usually only valid for the specific conditions upon which the derivation of the model is based upon [V1]. Moreover, Sittipunt [S1] indicated that most of the previous works in the finite element analysis of the behavior of RC walls are concentrated on the development of the material models that could reproduce experimental results and few research studies have used the finite element method to investigate behavior of RC walls other than that of the specimens tested in the laboratory. Therefore, general-purpose microscopic models are needed to be developed to describe the detailed local behavior of RC structural walls. With the development of the nonlinear structural analysis and nonlinear constitutive laws of material, the finite element method (FEM) is now a powerful means to yield detailed information on the behavior of RC structural walls, such as stress-strain relationships in concrete and reinforcing bars, deflected shapes, and crack patterns, which cannot be obtained from other analytical methods such as truss models and macroscopic models.

At present, numerous numerical modeling works for nonlinear finite element analyses of RC structural walls have been carried out but most of them were concentrated on its behavior under monotonic loading [K1-2, P1, N1] due to the complexities in the cyclic modeling of RC composite material after cracking of concrete. Several relevant studies have been carried out [E1, K3, K6, S1-2] in the analysis of actual RC structural walls regards microscopic models describing the behavior of RC under cyclic loading. Among them, the fixed crack model [E1, K6, S2] is popularly adopted for the modeling

of concrete because of its computational convenience and its capability of representing the physical nature of cracking in RC walls. However, it should be noted that the fixed crack model has a limitation in describing the rotation of cracks induced by the interface shear along the cracking surface when a RC structural wall is subjected to cyclic loadings [K3]. In this study, the rotating crack approach based on total strain is applied to the constitutive modeling of tensile and compressive behavior of concrete for the purpose of better understanding of shear effects of RC structural walls. In addition, although several experimental and analytical studies [M2-3, P3, T1] on the structural components with construction joints which may cause sliding shear failure have been investigated, research related to numerical modeling for such structural components, especially shear walls, is rather limited. In this study, the nonlinear structural interface element based on Mohr-coulomb friction criterion is adopted to aid in better understanding of the sliding behavior of such walls.

This study presents finite element analytical models for RC structural walls subjected to in-plane cyclic loadings. The proposed models are validated through comparison of the analytical results with extensive experimental data at global and local response levels with the aid of the finite element computer program DIANA 8.1 [D1]. By comparisons, this study shows that with proper calibration of material parameters and implementation of cyclic constitutive relationships, the proposed finite element models are effective in predicting the nonlinear behavior of RC structural walls and will play an important role in the ongoing research. These analytical models of the finite element method are then further used to explore in detail the influence of several critical parameters: axial loads, longitudinal reinforcements in the wall boundary elements, aspect ratio, area of boundary columns and the presence of construction joints at the wall base on the seismic behavior of walls.

5.2 Description of Finite Element Models

The computer program DIANA 8.1 (*Displacement method Analyser*, version 8.1) is a general purpose finite element code including nonlinear structural analysis of RC structures, based on the displacement method. The program has been developed at the TNO Building and Construction Research in Netherlands [D1] since 1972 with appealing capabilities in the field of concrete where excellent material models are available.

Reinforced concrete (RC) behaves as a composite structure under load and after concrete cracking, it shows complicated nonlinear behavior such as bond action between reinforcement and concrete, aggregate interlock along the crack interface and compressive deterioration of cracked concrete. To effectively describe the complicated nonlinear behavior of RC structures, the accurate modeling of the material properties becomes essential. In this finite element analysis computer program DIANA 8.1, four types of nonlinear material properties including the concrete, reinforcement and bondages between them as well as structural interface are to be provided.

5.2.1 Constitutive Models for Concrete

Since concrete is weak in tension, tensile cracking, which is one of the most important reasons for nonlinearities in RC, can have a significant effect on the behavior of most RC members, even at an early stage of loading. As a result, proper crack modeling is crucial to the success of the concrete model that generally includes the reasonable definitions of three basic components: crack representation, crack initiation and propagation, and the constitutive relationship for cracked concrete. In the following

sections, different approaches that have been used to define the three basic components would be discussed with the emphasis on those adopted in this study.

1) Crack representation

In the finite element analysis, stress and strain are assumed to be continuous within one finite element. However, when concrete cracks, discontinuities in stress and strain occur in the concrete matrix. To incorporate these discontinuities into the concrete model, there are two commonly used approaches to represent cracks: discrete crack model and smeared crack model. The discrete crack model represents cracks as a separation of nodes along element boundaries. The post-cracking behavior, such as tension stiffening, aggregate interlock, and dowel action, can be incorporated into the model by using linkage elements to connect the separated nodes. Although this model realistically represents the discontinuities in stress and strain across cracks, cracking can occur only along element boundaries that introduces bias into the finite element solution. Moreover, once the separation of the nodes has occurred, crack closing and reopening needs to be considered as a contact problem. This significantly complicates the finite element procedure, especially in the problems that involve cyclic loading [S1].

In the smeared crack model, the stress-strain discontinuities across the cracks are averaged over the element in the vicinity of the cracks and thus the stress-strain relationship of cracked concrete can still be described in a continuous manner. Since cracks at each integration point are considered separately, cracking can occur in any direction and multiple cracks are also allowed at each integration point. This model is found to be suitable for modeling RC members with distributed crack patterns because

it represents cracks as being finely spaced. However, for RC members in which one or few large cracks dominate the response, the discrete crack model might be more appropriate.

For RC structural walls, behavior of cracks are rather more important than the development of individual crack. For this purpose, the smeared crack model, in which a finite region containing several cracks and reinforcing bars are considered to be a continuum, is quite adequate to describe a RC element. On the other hand, reality is that local discontinuities, such as sliding along the joint plane, can take place due to the presence of construction joints at the wall base. To take this effect into consideration, the introduction of the discrete crack model becomes necessary and would be discussed in the following constitutive relationship of structural interface.

2) Crack initiation and propagation

This study adopts a strength criterion for crack initiation that concrete cracking occurs as the principal tensile stress violates the maximum stress condition. After a crack has formed, the fracture mechanics criterion for the smeared crack model is applied in this investigation, to determine crack propagation. In this approach, each crack is modeled by a one-element wide band of concrete elements. The cracks propagate to the next element at the tip of the crack band when the calculated energy release rate of the crack band exceeds the critical value, which depends on the fracture energy, G_f of concrete.

3) Constitutive modeling of cracked concrete

The finite element computer program, DIANA 8.1, offers a wide range of material

models for analysis of nonlinear concrete behavior. The models usually comprise tension softening, compression softening, and crack closing and reopening which are the major aspects of the inelastic behavior in concrete. In this study, total strain based crack models, which describe the tensile and compressive behavior of concrete with a single stress-strain relationship, is applied to simulate the nonlinear behavior of the concrete. This particular crack model can be used by either a rotating or fixed crack situation according to the method of determining a crack direction. The rotating crack model evaluates the stress-strain relationships in the principal directions of strain vector and no shear transfer model is needed in this approach since no shear stress appears in continually updated principal planes. Whereas the fixed crack model assumes that the crack is fixed once it is generated, in this model the shear transfer model is required. The fixed crack model is popularly adopted in numerical modeling of concrete cracking due to its computational convenience and more clearly defines the physical nature of concrete cracking. However, it also should be noted that the fixed crack model has a limitation in describing the rotation of cracks induced by the interface shear along the cracking surface when a RC structural wall is subjected to cyclic loadings [K3]. Thus, in this study, the rotating crack approach based on total strain is applied to the constitutive modeling of tensile and compressive behavior of concrete.

To simulate the tension softening effect of concrete after its cracking, the finite element software, DIANA 8.1, offers a range of softening models, such as linear tension softening, multilinear tension softening and nonlinear tension softening models. In this study, the nonlinear tension softening model proposed by Hordijk, Cornelissen and Reinhardt [D1] would be adopted as shown in Fig. 5.1. This model proposed an expression for the softening behavior of concrete which results in a crack stress equal to zero at a ultimate crack strain, $\epsilon_{nn,ult}^{cr}$ as expressed in Eq. (5.1).

$$\epsilon_{m,ult}^{cr} = 5.136 \frac{G_f}{h \cdot f_t} \quad (5.1)$$

The tensile strength, f_t (N/mm^2), tensile fracture energy, G_f ($N \cdot mm/mm^2$) and crack band width, h (mm) as shown in Fig. 5.1 are calculated in terms of the European CEB-FIP Model Code 1990 [C1]:

$$f_t = 1.4 \left(\frac{f_c'}{10} \right)^{\frac{2}{3}} \quad (5.2)$$

$$G_f = G_{f0} \left(\frac{f_c'}{10} \right)^{0.7} \quad (5.3)$$

The value of G_{f0} ($N \cdot mm/mm^2$) relates to the maximum aggregate size as listed in Table 5.1. Crack band width, h (mm) is the square root of the area of the element.

Concrete in compression is simulated with commonly used parabolic compression model as shown in Fig. 5.1 due to its ability to consider both the hardening and softening behavior of concrete. As indicated in Fig. 5.1, the parabolic compression model is in connection with the concrete compressive strength, f_c' , crack band width, h and compressive fracture energy, G_c which the value is usually defined by $150G_f$ [C1]. Moreover, it should be mentioned that concrete subjected to compressive stresses shows a pressure-dependent behavior, i.e., strength and ductility increase with increasing isotropic stress. In such a case, the compressive stress-strain relationship should be modified to incorporate the effects of the lateral confinement. In order to simulate this behavior, the parameters of the compressive stress-strain function in the

computer program DIANA 8.1 are determined with a failure function which gives the compressive stress as a function of the confining stresses in the lateral directions. The effect of lateral confinement on the strength and ductility of the concrete is clearly described in the literature [D1] by means of considering the corresponding parameters. In the current implementation of the computer program DIANA 8.1, secant reload and unload responses is applied to concrete in tension, while linear reload and unload responses with an initial stiffness same as that concrete in tension is used for concrete in compression.

5.2.2 Constitutive Model for Reinforcing Bars in Concrete

When modeling the reinforcing steel, bar yielding and strain-hardening effects must be considered as these effects are the major sources of energy dissipation in RC structures. In the finite element analytical program, DIANA 8.1, two methods for modeling reinforcing bars in concrete are available. The first method is called embedded reinforcement which as its name suggests, denotes that the reinforcement is embedded within the concrete. The strains of such reinforcement are obtained from the displacement field of concrete and so a perfect bond is assumed between the reinforcement and concrete. The second method applies discrete elements such as truss elements to model reinforcing bars in concrete which allows for the consideration of bond-slip behaviors between the reinforcement and surrounding concrete by adding interface elements between the reinforcement and the concrete. In this study, the embedded reinforcement, which allows the lines of the reinforcement to deviate from the lines of the mesh, are used to model the transverse reinforcement in wall boundary elements. Whereas, truss elements are applied to model other reinforcing bars in concrete with perfect bond as the effect of bending stiffness and dowel action of a

reinforcing bar on the behavior of low-rise structural walls is not very large. The constitutive behavior of the reinforcing bars herein is simulated by an elasto-plastic material model with strain hardening by employing the Von Mises yield criterion [D1].

5.2.3 Constitutive Model for Structural Interface

The computer program, DIANA 8.1, supplies several structural interface models by setting a nonlinear relation between tractions (normal/shear) and relative displacements across the interface to simulate the interface behavior such as bond-slip, crack dilatancy and friction. The general constitutive relation is assumed to be incrementally linear

$$\dot{t} = D \cdot \Delta \dot{u} \quad (5.4)$$

where t is the traction vector, Δu is the vector with the relative displacements, and D is the tangential stiffness matrix.

To simulate the behavior of existing construction joints between two parts of the structural walls, shear friction theory as shown in Fig. 5.2 is adopted herein as its suitable application for this case [D1]. In the shear friction hypothesis as shown in Fig. 5.2, the roughness is visualized as a series of frictionless fine saw-tooth ramps having a slope of $\tan \phi$. Assuming sliding along the failure plane m-m, and simple Coulomb friction, the shear force, V , required to produce sliding is equal to μP , where μ is the friction coefficient between the two elements and P is the clamping force perpendicular to the sliding plane. The roughness of crack m-m will create a separation δ between the two halves as shown in Fig. 5.2(b). If reinforcement is placed across

the interface, the separation will develop tension T in the reinforcement. The tension provides an external clamping force on the concrete resulting in compression across the interface of equal magnitude. Tests revealed that the separation is usually sufficient to yield the reinforcement crossing the crack. In this study the Coulomb friction model as shown in Fig. 5.3, suitable application for the shear friction theory, is adopted in the modeling of existing construction joints between two parts of the structural walls. As shown in Fig. 5.3, the Coulomb friction model based on the shear friction hypothesis presented is basically given by the yield surface

$$f = \sqrt{t_t^2} + t_n \tan \phi(k) - c(k) = 0 \quad (5.5)$$

and the plastic potential surface:

$$g = \sqrt{t_t^2} + t_n \tan \psi \quad (5.6)$$

where c denotes the cohesion, ψ is dilatancy angle which accounts for crack dilatancy effect and here equals to friction angle, ϕ for the consideration of associated plasticity [D1]. The detailed constitutive law for the model is presented in the DIANA manual [D1].

In the simulation of the structural interface, the Coulomb friction criterion is extended with a gap criterion: assuming that a gap arises if the tensile traction, t_n normal to the interface exceeds a certain value which corresponds to a concrete tensile strength, f_t . After the gap formation, the tensile traction, t_n is reduced to zero immediately and

shear retention according to the aggregate interlock relation of Walraven and Reinhardt [W1] is adopted to simulate the shear transfer mechanism along the structural interface. The response diagram for this model is shown in Fig. 5.4.

In accordance with ACI 318 code provisions [A1], which suggests the use of the value 1.4 as the friction coefficient for a crack in monolithic concrete, 1.0 for interfaces intentionally roughened and 0.6 for the interfaces which are not intentionally roughened. The specimens tested in this study are representative of poor construction practices since no specific measures were taken regarding curing at the construction joints. For this case the friction coefficient of 0.6 which takes the effect of shear resistance due to dowel action of the reinforcement into account is considered to be appropriate [A1] in the analysis of construction joints not intentionally roughened at the surface of the normal weight concrete.

5.3 Applications of the Finite Element Models

In the analysis, the developed concrete constitutive law is a total strain rotating crack model with nonlinear tension softening according to Hordijk *et al* [D1] and the parabolic compression as shown in Fig. 5.1. Eight-node quadrilateral isoparametric plane stress elements, QU8 (CQ16m), based on quadratic interpolation and 3×3 Gauss integration scheme are used to model concrete. The reinforcing bar is simulated as Von Mises plastic material and the strain hardening rule, developed according to the uniaxial test of bare bars, is applied. Embedded reinforcement is used to model the transverse reinforcements in wall boundary elements, whereas other bars is assumed to be a truss element with two nodes and perfect bond between reinforcement and

concrete is assumed. Six-node interface element between two lines in a two-dimensional configuration is used to simulate the interface behavior. The element is based on quadratic interpolation and the default 4-point Newton-Cotes integration scheme is applied. The mesh of structural walls is generally divided into four zones: the wall web, wall flanges, top and base beams. The top and base beam of the wall is considered to be elastic and rigid during the modeling. The base of the wall is assumed to be fully anchored against horizontal and vertical movements.

The horizontal cyclic loadings are assumed to act at the center location of the top beam which is considered to be rigid to uniformly distribute the lateral load to the entire cross-section of RC structural walls. The loading history of each specimen is similar to its experiment equivalent, but only one cycle is applied at each pre-defined drift ratio. In the current implementation of the computer program, DIANA 8.1, the behavior of unloading and reloading is modeled differently with secant unloading as shown in Fig. 5.1 [D1]. Moreover, in this nonlinear finite element analysis, regular Newton-Raphson iteration method in which the tangential stiffness matrix is evaluated for every iteration is chosen as an iterative procedure since a few iterations to converge to the final solution is needed.

5.3.1 Verification of the Finite Element Models

Before proceeding with an extensive parametric study using the nonlinear finite element analysis, it is necessary to verify the reliability of the proposed finite element models. For this purpose, two RC low-rise structural walls previously tested are selected. The first RC low-rise structural wall, Unit 1.0 with an aspect ratio of 1.0, was tested by Mestyanek [M1]. The tested wall eventually failed in shear due to diagonal

tension and thus is selected as a typical model to demonstrate shear effects. The second selected structural wall to validate the finite element models is Specimen S-F1 with an aspect ratio of 1.3 tested by Wu [W2]. The contribution of the flexure component to total displacement for this specimen is higher than 70% at the ultimate stage and it is, therefore, natural to conclude that the wall is mainly dominated by flexure. Table 5.2 lists the material properties of concrete and reinforcing bars according to the measured values as well as reinforcement ratios of the two selected specimens. Figs. 5.5 and 5.6 depict the overall dimensions and reinforcement layout of the tested specimens, Unit 1.0 and Specimen S-F1, respectively.

With exactly the same geometrical configuration and dimensions as the tested specimens, the finite element idealizations of Unit 1.0 and Specimen S-F1 are shown in Figs. 5.7 and 5.8, respectively. In these figures, deformed shapes of both specimens at a drift ratio of 1.0% are also presented to illustrate the effect of shear/flexure deformations on the wall performance. Meanwhile, the experimental and analytical lateral load – displacement relationships of Unit 1.0 and Specimen S-F1 are illustrated in Figs. 5.9 and 5.10, respectively. From Figs. 5.7 and 5.9, it can be seen clearly that significant tensile and shear straining has occurred in the wall web due to the insufficient provisions of shear reinforcements for Unit 1.0, while from Figs. 5.8 and 5.10, it is observed that the response of Specimen S-F1 is dominated by flexure. It can thus be naturally concluded that the observed analytical responses for both studied specimens agree well with the respective test results.

Table 5.3 presents the comparisons of finite element predictions and test results in terms of peak loads obtained and its associated displacements for both specimens. As observed at Columns (6) and (7) in Table 5.3, for Unit 1.0 and Specimen S-F1, the ratio

of tested-to-predicted peak loads, $F_{p,t} / F_{p,p}$ is obtained to be 0.96 and 1.11, respectively, while the ratio of tested-to-predicted associated displacements, $\Delta_{p,t} / \Delta_{p,p}$ is 0.96 and 1.01, respectively. This comparison indicates that the finite element analytical results correlate well with the experimental ones in terms of strength and deformation capacities for both specimens. However, for Specimen S-F1 the pinching degree of the analytical results as shown in Fig. 5.10 is slightly overestimated which may be the production of the assumed perfect-bond between the reinforcement and concrete. In practice, the presence of a bond slip may have a pronounced effect on the pinching of the specimen. In general, it can be concluded that the proposed finite element models implemented in the finite element analysis can simulate satisfactorily the behavior of RC low-rise structural walls under reversed cyclic loadings.

5.3.2 Numerical Investigations of Specimens Tested

Previous chapters presented the experimental results of eight RC structural walls, including both global responses, such as load-displacement relationships, and local responses, such as rebar strain history. Based on those results, the following numerical investigations are carried out to fulfill two main purposes. Firstly, the investigations are intended to verify the reliability of the proposed finite element models under different design parameters such as axial loads and aspect ratios, especially the structural interface model for the tested walls. Secondly, the investigations are extended by comparing the numerical local response of the structural walls tested with the experimental ones for which offered more insights on the local behavior of all tested specimens.

The concrete and steel material properties adopted for these nonlinear finite element investigations of the total eight specimens are same as those used in the testing. The detailed concrete properties are listed in Table 5.4 and steel properties as displayed in Fig. 2.1 at chapter two are employed by these investigations. In the case of structural walls with construction joints at the wall base, Coulomb friction model, with a gap criterion and shear retention according to the aggregate interlock relation of Walraven and Reinhardt [W1], is adopted to simulate the shear transfer mechanism along the structural interface. In the following investigations a friction coefficient of 0.6, which takes the effect of shear resistance due to dowel action of the reinforcement into account, is considered to be appropriate [A1] in the analysis of construction joints not intentionally roughened at the surface of normal weight concrete.

In the following sections the numerical results, including the respective global and local responses of selected specimens, according to the proposed finite element models are compared with experimental ones. After comparison with experiment results, a parametric study is carried out in this research for RC structural walls to establish the significance of several critical parameters, such as axial loads, longitudinal reinforcements in the wall boundary elements, aspect ratio, area of boundary columns and the presence of construction joints at the wall base. The effect of these critical parameters on structural performances such as strength and deformation capacity, stiffness characteristic, energy dissipation and equivalent damping of RC structural walls is investigated as follows.

5.3.2.1 Predicted Global Response of Specimens Tested

The idealized finite element meshes for total eight specimens are illustrated in Fig. 5.11

and have exactly the same geometry configuration and dimensions as the respective tested specimens. The overstriking black continuous line at the wall base as displayed in Figs. 5.11(c) and 5.11(f) indicates the application of structural interface models to associated specimens. Embedded reinforcement which allows the lines of the reinforcement to deviate from the lines of the mesh are used to model the transverse reinforcements in boundary elements for all specimens tested. Whereas, truss elements located along the sides of element meshes are applied to model other reinforcing bars in concrete with perfect bond.

Global behavior is presented in terms of hysteretic loops: lateral load – displacement relationships of specimens studied. Fig. 5.12 shows the comparison results between the experimental and analytical lateral load – top displacement relationships for all eight specimens. In general, analytical results show a good correlation with the experimental ones in terms of strength capacity, deformation characteristics and energy dissipation for all specimens. This firstly suggests that the structural interface model incorporated in the proposed finite element models during numerical investigations is effective in predicting the behavior of RC structural walls with construction joints at the base. Secondly, it can be concluded that the proposed finite element models are reliable for RC structural walls under different axial loading levels and aspect ratios. However, by comparing experimental initial stiffness, the analytical initial stiffness of specimens studied is found to be overestimated which may due to the assumptions made as fully fixed boundary conditions, perfect bond between the reinforcement and concrete, and loads applied exactly at the wall center line during the analysis.

5.3.2.2 Predicted Local Response of Specimens Tested

Numerical investigations of the wall local responses are carried out by outputting the strain distributions along the selected bars for Specimens LW1, LW2 and MW1. In this study strain distributions along two types of selected bars, longitudinal and horizontal bars, are investigated respectively for the selected specimens. Moreover, for the purpose of better understanding the strain development tendency as tests progressed, four levels of drift, 0.1%, 0.25%, 0.5% and 1.0% corresponding to the stages of the pre-cracking, first-yield, post-yield and ultimate respectively, are considered for each selected specimen. At each drift level, both the experimental and numerical strains in selected longitudinal/horizontal bars of the studied specimens are presented to calibrate their correlation.

Fig. 5.13 depicts the experimental and numerical longitudinal strains in selected vertical bars along wall length of the specified specimens. As shown in Fig. 5.13, for each specimen under the negative loading, the strain distributions along the wall length is obtained at two different wall heights: 50 mm and 550 mm measured from the wall base. These wall heights correspond to exactly the same position of strain gauges attached to the selected longitudinal bars in the tests and at each wall height, eight longitudinal strains obtained from eight different selected vertical bars are presented to indicate the strain distributions along the whole wall length. It is found from Fig. 5.13 that at low drift levels the experimental and numerical longitudinal strains in selected vertical bars agree well and distribute almost linearly along the wall length which indicates that the Navier-Bernoulli assumption of plane-sections remaining plane can be applied to all three selected specimens up to a drift ratio of 0.5%. However, as displacing the specimens to a drift level of 1.0%, a significant increase of the values of

longitudinal strains is observed in selected vertical bars and the plane sections do not remain plane at the wall base. At this drift level the numerical strains in longitudinal bars for selected specimens are observed to be slightly higher than those in the tests, but in general good correlation between the numerical longitudinal strains and the experimental ones in vertical bars is observed at all four drift levels.

Fig. 5.14 depicts the experimental and numerical strain distributions along horizontal bars of the three selected specimens under both positive and negative loading directions. At each specimen two horizontal bars, R bar and T bar, located at two different levels of the wall height: 250 mm and 750 mm measured from the wall base, respectively are selected to distribute the strains at four different locations along the selected horizontal bar as displayed in Fig. 5.14. The strain locations exactly correspond to the locations of strain gauges attached to the specified horizontal bars in the tests. In general, larger values of horizontal strains along the diagonal struts than those in other positions are observed as shown in Fig. 5.14 for both experimental and numerical strains in the two specified horizontal bars for selected specimens. The observed good correlation between the experimental and numerical results further provides the verification of the effectiveness of the proposed finite element models in predicting the local responses of walls studied.

5.3.3 Parametric Study of Squat Structural Walls

In the past decades, most of the parametric works in the finite element analysis of RC structural walls were concentrated on the development of the material models that could reproduce experimental results and few researchers used the finite element method to investigate behavior of RC walls other than that of the specimens tested in

the laboratory [S1]. According to currently available information at hand, two main parametric studies using the finite element method were carried out up to date to study the effects of different design parameters on the response of RC structural walls. The first study was conducted in Japan by Mikame *et al* [M4] who investigated the effects of several design parameters such as different reinforcement ratios, vertical axial stress, compressive strength of concrete, confinement of column, and openings on the behavior of heavily reinforced structural walls. The second was undertaken at University of Ottawa, Canada by Nasir N. [N3] who investigated the sensitivity of selected design parameters, such as axial loads and aspect ratios etc, on strength and ductility of eighteen RC structural walls. However, both studies were carried out for RC low-rise structural walls subjected to static monotonic loading. This leads to difficulties in the evaluation of several critical seismic performances, such as energy dissipation and equivalent damping, which can only be assessed by means of modeling the structural walls under cyclic loadings.

Accordingly, for the purpose of evaluating the effect of different design parameters on the wall seismic performances, it is necessary to carry out an extensive parametric study for RC low-rise walls under cyclic loadings. In this study, five critical design parameters with their description and investigated ranges are listed in Table 5.4. The effects of these investigated parameters on the wall seismic performance such as the secant stiffness, K_{eq} , energy dissipation capacity, A_h and equivalent damping factor, h_{eq} as shown in Fig. 5.15 are investigated. Note that the experimental and analytical equivalent damping factor, h_{eq} for walls studied as displayed in Fig. 5.15 is determined by

$$h_{eq} = \frac{1}{2\pi} \cdot \frac{A_h}{F_m \Delta_m} \quad (5.7)$$

where A_h is the energy dissipation calculated from the area enclosed by the load-displacement loops as shown in Fig. 5.15, F_m , Δ_m is the peak lateral load and top displacement of the loop, respectively.

In the following parametric studies, the tested walls as presented in the previous chapters are selected as reference walls. The overall dimension and reinforcement details of the studied RC walls are kept the same as those tested except those specified in Table 5.4. The concrete uniaxial compressive strength, f'_c of 40 MPa was selected, with an initial tangent modulus of elasticity equal to 29,900 MPa. A concrete tensile strength of 3.15 MPa was assumed and a constant Poisson's ratio of 0.2 was selected for all specimens studied. The steel properties used in the parametric studies are the same as those in the tests.

In the following analysis, the verified finite element material models would be adopted to fulfill this purpose. The final analysis results will be presented together with the experimental ones to aid in better understanding of the effect of different design parameters on the seismic performances of the structural walls studied.

5.3.3.1 Effect of Axial Loading

The investigations on effects of axial load are carried out for eight cases by changing the axial load ratio, $N/(f'_c A_g)$ from 0.00 to 0.15. A series of two RC walls with

aspect ratios fixed to be 1.125 and 1.625, respectively are investigated by selecting the tested Specimens LW1 and MW1 as reference walls in order to compare the analytical results with experimental ones in terms of seismic performance. The reinforcement detail and overall dimensions of the walls studied are kept to be the same as those for Specimens LW1 and MW1.

5.3.3.1.1 Effect of axial loading on wall strength

Figs. 5.16(a) and 5.16(b) demonstrate the analytical and experimental backbone curves of the lateral load – top displacement for two series of walls with aspect ratios of 1.125 and 1.625, respectively. In general, the presence of axial loading significantly increases the wall strength capacity with the augment of axial loads up to a ratio of 0.15 for both two series of RC walls. Moreover, as shown in Fig. 5.16, the lateral load – top displacement relationship show tri-linear for both two series of RC walls subjected to axial loads up to a ratio of 0.05 and a drift capacity of around 1.0% can be achieved for all walls. When axial load ratio reaches 0.10 or above, an arc-shaped relationship is observed and the wall drift capacity tends to decrease with the increase of axial loads.

The contribution of axial load ratios to the increase of wall strength is illustrated in Fig. 5.17 where $P_{i,max}$ and $P_{0,max}$ is the maximum strength for RC walls with different levels of axial loads and without axial loads, respectively. In this figure, the analytical results corresponding to the two series of RC walls with aspect ratios of 1.125 and 1.625, respectively are outputted together with experimental ones. It can be seen from the figure that the analytical rate of increase of the wall strength with the augment of axial load ratio from 0.00 to 0.05 correlates well with experimental results, and is

obtained to be 0.390 for RC walls with an aspect ratio of 1.125. However, with the increase of the wall axial load ratio from 0.10 to 0.15, the rate of increase of wall strength tends to be less, and is observed to be 0.181. Such variation trends can also be observed for RC walls with an aspect ratio of 1.625 for which the axial loads contribution to the wall strength decreases from 0.278 to 0.153 when the RC walls subjected to a higher level of axial loads. This indicates that the effect of axial load on the wall strength is more significant for RC walls under lower level of axial load (from 0.0 to 0.05) than higher level of axial load (from 0.10 to 0.15).

Moreover, the rate of increase of the wall strength, 0.390 with the augment of axial load ratio from 0.00 to 0.05 for RC walls with an aspect ratio of 1.125 is larger than that for RC walls with an aspect ratio of 1.625 as the value is observed to be 0.278. Similar variation observation can be extended to RC walls subjected to added axial load ratios from 0.10 to 0.15 as the rate of increase of wall strength for RC walls with aspect ratios of 1.125 and 1.625, is obtained to be 0.181 and 0.153, respectively. This suggests that the contribution of axial load to the wall strength for RC walls with a lower aspect ratio is more significant.

5.3.3.1.2 Effect of axial loading on secant stiffness

The representation of secant stiffness for RC walls under cyclic loading is shown in Fig. 5.15. Based on this representation, the variation of the experimental and analytical secant stiffness with drift ratios of RC walls subjected to four different levels of axial loads are presented in Fig. 5.18. In general, experimental secant stiffness agrees well

with analytical ones and both experimental and analytical wall secant stiffness degrades significantly with the increase of wall drift ratios. However, under an axial load ratio of 0.05, the analytical secant stiffness is observed to be overestimated which becomes more significant for both two series of walls at a lower wall drift ratios of up to 0.33%. This may be due to the assumptions of fully fixed boundary conditions, perfect bond between the reinforcement and concrete, and loads applied exactly at the wall center line during the analysis. Moreover, it can also be seen from Fig. 5.18 that the secant stiffness of RC walls with same drift ratios generally increases with the addition of axial loads at each drift level. However, this axial load contribution to the increase of secant stiffness reduces with the movement of top wall drift from a lower to higher level. At a lower wall drift ratio, the analytical secant stiffness of RC walls increases significantly with the added axial loads and a large gap between the secant stiffness of RC walls under different levels of axial loads is observed at same drift levels. However, at a higher wall drift ratio, the secant stiffness of a wall under different axial loadings becomes closer.

The contribution of axial loads to the wall secant stiffness at three different drift ratios, 0.1%, 0.33% and 1.0%, are shown in Fig. 5.19 where $K_{i,a}$ and $K_{0,a}$ is the analytical secant stiffness with different levels of axial loads and without axial loads, respectively. In the figure, the rate of increase of secant stiffness is determined by $(K_{i,a} - K_{0,a}) / K_{0,a}$. It can be observed that the rate of increase of secant stiffness generally decreases with the added drift ratios of RC walls studied. At a drift ratio of 0.1%, the increase rate of secant stiffness for RC wall with an aspect ratio of 1.125, increases from 109% to 246% with the augment of axial loads from 0.05 to 0.15, while at a drift ratio of 0.33%, the corresponding values reduces to be 77% and 188%. Moreover, the contribution of axial loads to wall secant stiffness is more significant for

RC walls with axial loads increasing from 0.0 to 0.05 than that for RC walls with axial loads increasing from 0.10 to 0.15. With added axial loads from 0.0 to 0.05, the contribution of axial loads is 109% and 99% for RC walls with aspect ratios of 1.125 and 1.625 respectively, while with the augment of axial loads from 0.10 to 0.15, 60% and 21% of this axial load contribution to secant stiffness is observed for all studied RC walls at a drift ratio of 0.1%. Similar decreasing trend of the contribution of axial loads to secant stiffness with the augment of axial loads can also be observed for all studied RC walls at a drift ratio of 0.33%.

5.3.3.1.3 Effect of axial loading on energy dissipation

Fig. 5.20 displays the variation of energy dissipation capacity with the drift ratios for both two series of walls under four different levels of axial loads. In general, the energy dissipation capacity increases with the added drift ratios for RC walls under all levels of axial loads. However, the effect of axial load on energy dissipation capacity is different for walls at a low drift ratio as compared to those at a high drift ratio. For wall drift ratios of up to 0.33%, the energy dissipation almost remains to be same for all RC walls under four different axial loads levels, however, with the increase of drift ratios from 0.33% to 1.0%, it is observed to increase almost linearly as shown in Fig. 5.20.

This variation trend can also be clearly seen in Fig. 5.21 which shows the contribution of axial load ratios to energy dissipation for all RC walls studied. In the figure, the continuous and dashed lines represent for RC walls with aspect ratios of 1.125 and 1.625, respectively. In the figure, the percentage increase of energy dissipation for RC walls with drift ratios of 0.50% or above is obtained by $(A_{h,i} - A_{h,0}) / A_{h,0}$ where $A_{h,i}$

and $A_{h,0}$ are the dissipated energy corresponding to RC walls under axial load ratios of 0.05 and 0.00, respectively. As displayed in this figure, with the added axial loads from 0.00 to 0.05, the rate of increase of energy dissipation for RC walls at a drift ratio of 0.50% is obtained to be 13% and 21%, while at a drift ratio of 1.0% this rate increases to 56% and 31% for RC walls with aspect ratios of 1.125 and 1.625, respectively. This suggests that the presence of axial loads plays a beneficial effect on the wall energy dissipation capacity at a high drift ratio, while at a low drift ratio (up to 0.33%) this effect is rather negligible.

5.3.3.1.4 Effect of axial loading on equivalent damping

The effect of axial load on equivalent damping of walls studied is shown in Fig. 5.22. As indicated in the figure, the equivalent damping of RC walls studied increases with the added drift ratios in general. At a low drift ratio (up to 0.33%), analytical equivalent damping for RC walls under all four levels of axial loads, increases slightly but remains to be low, whereas after that, the equivalent damping is observed to increase significantly for RC walls under axial load ratios from 0.00 to 0.05.

The contribution of axial loads to the equivalent damping for RC walls studied are also presented and illustrated in Fig. 5.23. In the figure, the continuous and dashed lines represent for RC walls with aspect ratios of 1.125 and 1.625, respectively. The percentage decrease of equivalent damping for RC walls with drift ratios of 0.50% or above is obtained by $(h_{eq,i} - h_{eq,0}) / h_{eq,0}$ where $h_{eq,i}$ and $h_{eq,0}$ are the equivalent damping corresponding to RC walls under axial load ratios of 0.05 and 0.00,

respectively. In general, the equivalent damping for both two groups of walls studied decreases with the added axial loading as shown in Fig. 5.23. With the added axial loads from 0.00 to 0.05, the percentage decrease of equivalent damping for RC walls at a drift ratio of 0.50% is obtained to be 42% and 35%, while at a drift ratio of 1.0% this decrease rate is observed to be 1.4% and 22% for RC walls with aspect ratios of 1.125 and 1.625, respectively. This can be due to the decrease of degree of loop pinching with the added axial load ratio from 0.0 to 0.15 which indicates its beneficial effect on the control of pinching behavior.

5.3.3.2 Effect of Longitudinal Reinforcement Content in Boundary Element

The effect of longitudinal reinforcement content in boundary element, ρ_b , on wall strength is investigated for four cases by changing this content, from 0.7% to 4.2% as listed in Table 5.5. The tested RC wall, Specimen LW1 with an aspect ratio of 1.125, is selected as reference wall in order to compare the analytical results with experimental ones in terms of seismic performance. The overall dimensions of the walls studied are kept to be the same as that of Specimens LW1.

5.3.3.2.1 Effect of longitudinal reinforcement content on wall strength

Fig. 5.24 demonstrates the effect of longitudinal reinforcement content on the backbone curves of load-displacement loops. As shown in Fig. 5.24, the wall strength capacity increases significantly with the augment of longitudinal reinforcement content in wall boundaries and a drift capacity of around 1.0% can be achieved for all walls studied. Moreover, as shown in Fig. 5.24, the lateral load – top displacement relationship show

tri-linear for RC walls having longitudinal reinforcement content of 2.8% or above, while bilinear backbone curves is observed for RC walls with longitudinal reinforcement content less than 1.4%.

The contribution of longitudinal reinforcement content to the wall strength capacity is shown in Fig. 5.25 where $P_{0,max}$ and $P_{i,max}$ is the maximum strength for RC walls with longitudinal reinforcement content of 0.7% and one of the other three different content levels, respectively. As shown in the figure, with the added longitudinal reinforcement content in boundary elements from 0.7% to 4.2%, the analytical maximum strength increases almost linearly and the percentage increase of the maximum strength is observed to be 140% around.

5.3.3.2.2 Effect of longitudinal reinforcement content on secant stiffness

The variation of the experimental and analytical secant stiffness with the drift ratios of RC walls having four different ratios of longitudinal reinforcement in boundary element are shown in Fig. 5.26. In general, the experimental secant stiffness agrees well with the analytical ones and both experimental and analytical secant stiffness degrade significantly with the increase of the wall drift ratios. With the increase of wall drift ratios from 0.1% to 1.0%, the secant stiffness for RC wall with longitudinal reinforcement ratio of 0.7%, reduces from 93 kN/mm to 12 kN/mm (around 87% degradation) in the positive loading direction. For RC walls with same drift ratios, the secant stiffness increases with the added longitudinal reinforcement content in wall boundaries. With the augment of this reinforcement content from 0.7% to 4.2%, the secant stiffness for RC walls at a drift ratio of 0.1% increases by around 52% from 91 kN/mm to 139 kN/mm.

The contribution of longitudinal reinforcement content to the wall secant stiffness at three different drift ratios, 0.1%, 0.33% and 1.0%, are shown in Fig. 5.27 where $K_{0,a}$ and $K_{i,a}$ is the analytical secant stiffness corresponding to RC walls with longitudinal reinforcement content of 0.7% and one of the other three different reinforcement ratios, respectively. As shown in the figure, the rate of increase of secant stiffness increases more rapidly after the wall attained a drift ratio of 0.33% for all four RC walls studied. At a drift ratio of 0.1% the increase rate of secant stiffness for RC walls with longitudinal reinforcement ratio of 1.4% and 4.2%, increases by 1.5% and 52%, respectively. While at a drift ratio of 0.33% the increase rate of secant stiffness is observed to be 22% and 133% and at the final stage, it is obtained to be 26% and 144% for RC walls with longitudinal reinforcement content of 1.4% and 4.2%, respectively.

Moreover, the contribution of longitudinal reinforcement ratio to wall secant stiffness is observed to be almost the same for RC walls with reinforcement ratios increasing from 1.4% to 2.8% as that for RC walls with reinforcement ratios ranging from 2.8% to 4.2% at/after the attainment of a wall drift ratio of 0.33%. With added longitudinal reinforcement content from 1.4% to 2.8%, the contribution of this reinforcement content, represented by a slope between them, is 0.43 and 0.41 for RC walls at drift ratios of 0.33% and 1.0% respectively, while 0.36 and 0.43 of this reinforcement contribution to secant stiffness is observed for RC walls with longitudinal reinforcement content ranging from 2.8% to 4.2%. However, at a drift ratio of 0.1%, the contribution of this reinforcement content ranging from 1.4% to 2.8% and from 2.8% to 4.2% is obtained only to be 0.23 and 0.14, respectively which is less than that for RC walls at higher drift ratios. This indicates that the contribution of longitudinal reinforcement to the wall secant stiffness is more effective for walls at higher drift

ratios.

5.3.3.2.3 Effect of longitudinal reinforcement content on energy dissipation

Fig. 5.28 displays the variation of energy dissipation capacity with the drift ratios for RC walls under four different ratios of longitudinal reinforcement in boundary elements. In general, the energy dissipation capacity increases with the added drift ratios for all RC walls studied. At a drift ratio of 0.1%, the energy dissipated for RC walls with longitudinal reinforcement content of 0.7% is obtained to be 0.38 kNm while at drift ratios of 0.33% and 1.0%, it is approximately 0.66 kNm and 8.12 kNm, respectively. This indicates that the effect of longitudinal reinforcement on energy dissipation capacity is different for walls at a low drift ratio (up to 0.33%) as compared with that at a high drift ratio. For wall drift up to a ratio of 0.33%, the energy dissipation almost remains to be same for all RC walls under all four different longitudinal reinforcement contents, however, with the increase of drift ratios from 0.33% to 1.0%, it is observed to increase significantly as shown in Fig. 5.28.

This variation trend can also be clearly seen in Fig. 5.29 which shows the contribution of longitudinal reinforcement to energy dissipation for all the RC walls studied. In the figure, the percentage increase of energy dissipation for RC walls with drift ratios of 0.50% or above is obtained by $(A_{h,i} - A_{h,0}) / A_{h,0}$ where $A_{h,0}$ and $A_{h,i}$ are the energy dissipated corresponding to RC walls with longitudinal reinforcement content of 0.7% and one of other three contents (1.4%, 2.8% and 4.2%), respectively. As displayed in this figure, with the added longitudinal reinforcement contents from 0.7% to 4.2%, the

rate of increase of energy dissipation for RC walls at a drift ratio of 0.50% is obtained to be 56%, while at drift ratios of 0.67% and 1.0%, this rate increases to 32% and 51%, respectively. This suggests that at a high drift ratio (from 0.5% to 1.0%), the increase of longitudinal reinforcement in boundary elements have a significant effect on the wall energy dissipation capacity, while at a low drift ratio (up to 0.33%) this effect is rather negligible.

5.3.3.2.4 Effect of longitudinal reinforcement content on equivalent damping

The effect of longitudinal reinforcement on equivalent damping of walls studied is shown in Fig. 5.30. As indicated in the figure, the equivalent damping of RC walls studied increases with the added drift ratios in general. At a low drift up to ratio of 0.25% (prior to reinforcement yielding), analytical equivalent damping for RC walls under four different longitudinal reinforcement contents, decreases significantly and minimum equivalent damping (below 4%) is observed at a drift ratio of 0.25% except for RC wall with reinforcement content of 0.7% which it is observed to be above 4% at a drift ratio of 0.33%. Whereas after that, the equivalent damping is observed to increase significantly to the final stage (above 12%) for all RC walls studied.

The contribution of longitudinal reinforcement to the equivalent damping for RC walls studied are also presented and illustrated in Fig. 5.31. The percentage decrease of equivalent damping for RC walls with drift ratios of 0.50% or above is obtained by $(h_{eq,i} - h_{eq,0}) / h_{eq,0}$ where $h_{eq,0}$ and $h_{eq,i}$ are the equivalent damping corresponding to RC walls with longitudinal reinforcement content of 0.7% and one of other three contents (1.4%, 2.8% and 4.2%), respectively. In general, the equivalent damping for

RC walls studied decreases with the added longitudinal reinforcements as shown in Fig. 5.31. With the added longitudinal reinforcements content from 0.7% to 1.4% and 4.2%, the percentage decrease of equivalent damping for RC walls at a drift ratio of 0.50% is obtained to be 21% and 44% respectively, while at a drift ratio of 1.0% this decrease rate is observed to be 7% and 36%, respectively.

5.3.3.3 Effect of Boundary Columns

Previous research [P2, W2] indicates that sliding shear failure of boundary columns in walls can greatly impair their energy dissipation capacities. Also it is observed [W2] that RC walls with boundary columns can achieve higher strength, stiffness, and maintain similar drift levels when subjected to reversed cyclic loading. To better understand and clarify the influence of wall boundary columns on the global behavior of walls with cyclic loadings, a parametric study is carried out by varying the cross sections of boundary elements in four cases. This is achieved by changing the thickness of boundary columns, t_f from 120 mm (rectangular) to 1000 mm which leads to the ratio of the sectional area of columns to the total sectional area, A_c / A_t varying from 0.15 to 0.60 as listed in Table 5.5 where A_c and A_t are the cross section areas of boundary columns and the entire wall, respectively. The tested RC wall, Specimen LW1 with an aspect ratio of 1.125, is selected as a reference wall. The overall dimensions and the quantity of main bars in boundary columns of the RC studied walls are kept to be the same as that of Specimens LW1.

5.3.3.3.1 Effect of boundary columns on wall strength

Fig. 5.32 demonstrates the effect of boundary element shapes on the backbone curves of load-displacement loops. As shown in Fig. 5.32, the wall strength increases significantly with an increase of boundary column area ratios and a drift capacity of around 1.0% can be achieved for all walls. This contribution of boundary columns to the wall strength is also shown in Fig. 5.33 where $P_{0,max}$ and $P_{i,max}$ is the maximum strength for RC walls with a column area ratio of 0.15 and one of the other three different ratios, respectively. As shown in the figure, the percentage increase of the analytical maximum strength is observed to be 14.2% with the increase of area ratio from 0.15 to 0.60. Moreover, the trend of strength increases is observed to be more significant as column area ratios increase. With the change of area ratios from 0.15 to 0.30, the analytical maximum strength increases relatively to be 0.11, while the increasing values are obtained to be 0.21 and 0.63 corresponding to the area ratios ranging from 0.30 to 0.45 and 0.45 to 0.60, respectively.

5.3.3.3.2 Effect of boundary columns on secant stiffness

The variation of the experimental and analytical secant stiffness with the drift ratios of RC walls having four different column area ratios are shown in Fig. 5.34. In general, the experimental secant stiffness agrees well with the analytical ones. The wall secant stiffness degrades significantly with the increase of the wall drift ratios. With added wall drift ratios from 0.1% to 1.0%, the secant stiffness for RC wall with an area ratio of 0.15, reduces from 90 kN/mm to 15 kN/mm (around 83% degradation) in the positive loading direction. It can also be seen from the figure that the secant stiffness of RC walls with same drift ratios increases with the higher column area ratios. With the

increase of this ratio from 0.15 to 0.60, the secant stiffness for RC walls at a drift ratio of 0.1% increases by around 58% from 90 kN/mm to 142 kN/mm.

The contribution of boundary columns to the wall secant stiffness at three different drift ratios, 0.1%, 0.33% and 1.0%, are shown in Fig. 5.35 where $K_{0,a}$ and $K_{i,a}$ is the analytical secant stiffness corresponding to RC walls with an area ratio of 0.15 and one of other three different area ratios, respectively. As shown in the figure, the rate of increase of secant stiffness with the augment of column area ratios, decreases more rapidly after a wall drift ratio of 0.10% for all four studied RC walls. At a drift ratio of 0.1%, the increase rate of secant stiffness, for RC wall with area ratios of 0.45 and 0.60, increases by 29% and 64% respectively. While at a drift ratio of 0.33%, the increase rate of secant stiffness is observed to be 7% and 16% and at the final stage, it is observed to be 10% and 17% for RC walls with area ratios of 0.45 and 0.60, respectively.

5.3.3.3 Effect of boundary columns on energy dissipation

Fig. 5.36 displays the variation of the energy dissipation capacity with drift ratios for RC walls under four area ratios of boundary elements. In general, the energy dissipation capacity increases with the higher area ratios of boundary columns. At a drift ratio of 0.1%, the energy dissipated for RC walls with area ratio of 0.15 is obtained to be 0.20 kNm around while at an area ratio of 0.60, it is approximately 1.2 kNm. This variation trend can also be clearly seen in Fig. 5.37 which shows the contribution of boundary area ratios to energy dissipation for all RC walls studied. In

the figure, the percentage increase of energy dissipation for RC walls with drift ratios of 0.50% or above is obtained by $(A_{h,i} - A_{h,0}) / A_{h,0}$ where $A_{h,0}$ and $A_{h,i}$ are the dissipated energy corresponding to RC walls with boundary area ratio of 0.15 and one of other three area ratios (0.30, 0.45 and 0.60), respectively. As displayed in this figure, with the added area ratios from 0.15 to 0.30, the rate of increase of energy dissipation for RC walls at a drift ratio of 0.5% and 1.0% is obtained to be 0.0% and 6.0%, respectively. With boundary area ratios increasing from 0.30 to 0.45, the rate of increase of energy dissipation at drift ratios of 0.5% and 1.0% is 32% and 14%, respectively. This suggests that at a high area ratio, the increase of area ratios of boundary columns have a significant effect on the wall energy dissipation capacity, while at low area ratios (from 0.15 to 0.30), this effect is rather negligible.

5.3.3.3.4 Effect of boundary columns on equivalent damping

The effect of boundary columns on equivalent damping of walls studied is shown in Fig. 5.38. In general, the equivalent damping of walls increases with higher area ratios of boundary columns. At a low drift of up to a ratio of 0.25% (prior to reinforcement yielding), analytical equivalent damping for RC walls under four different area ratios, decreases significantly and minimum equivalent damping (below 10.0%) is observed at a drift ratio of 0.25% except for the RC wall with an area ratio of 0.60 which it is observed to be above 15% at a drift ratio of 0.33%. Whereas after that, the equivalent damping is observed to increase significantly till the final stage (above 15%) for all RC walls studied.

The contribution of boundary columns to the equivalent damping for RC walls studied

is presented in Fig. 5.39. The percentage increase of equivalent damping for RC walls with drift ratios of 0.67% and 1.0% is obtained by $(h_{eq,i} - h_{eq,i-1}) / h_{eq,i-1}$ where $h_{eq,i}$ and $h_{eq,i-1}$ are equivalent damping corresponding to current and previous area ratios, respectively. At drift ratios of 0.67%, with the added area ratios from 0.15 to 0.30, from 0.30 to 0.45, and from 0.45 to 0.60, the percentage increase of equivalent damping for RC walls is obtained to be 9.0%, 15.0%, and 38.0%, respectively. This suggests that the rate of increase of the equivalent damping becomes more significant with the added boundary area ratios with the observations that the largest rate obtained with the area ratio changing from 0.45 to 0.60 while the least value of increase rate for area ratios ranging from 0.15 to 0.30. A similar trend can be achieved for studied walls at a drift ratio of 1.0% which the largest (21%) and least (0.0%) increase rate of equivalent damping obtained corresponds to boundary area ratios ranging from 0.45 to 0.60 and 0.15 to 0.30, respectively. It indicates that larger boundary area ratios have a more significant effect on the equivalent damping.

5.3.3.4 Effect of Aspect Ratios

The effect of aspect ratios, H_w/L_w on wall structural performances is investigated for four cases by changing the aspect ratio from 0.5 to 2.0 as listed in Table 5.24. The tested RC wall, Specimen LW1 with an aspect ratio of 1.125, is selected as a reference wall in order to compare the analytical results with experimental ones in terms of seismic performance. Different wall aspect ratios, H_w/L_w are achieved by varying the wall height, H_w and keeping the wall length, L_w same as that of the reference

wall. The reinforcement details of the walls studied are kept the same as that of the reference wall.

5.3.3.4.1 Effect of aspect ratio on wall strength

Fig. 5.40 demonstrates the effect of wall aspect ratios on the backbone curves of load-displacement loops and the wall strength capacity is observed to decrease significantly with the augment of wall aspect ratios. As shown in the figure, the percentage decrease of the analytical maximum strength is observed to be 47.7%, 63.3% and 70.7% with the increase of aspect ratios from 0.50 to 1.125, 1.625 and 2.0, respectively. As shown in Fig. 5.40, a drift capacity of around 1.0% can be achieved for walls studied other than for the wall with an aspect ratio of 0.5 for which a drift ratio of 0.5% is achieved.

5.3.3.4.2 Effect of aspect ratios on secant stiffness

For all RC walls with four different aspect ratios, the variation of the experimental and analytical secant stiffness with the wall drift ratios is shown in Fig. 5.41. In general, the experimental secant stiffness agrees well with the analytical ones with the observations that wall secant stiffness degrades significantly with the increase of the wall drift ratios. With the augment of wall drift ratios, the rate of decrease of the secant stiffness for RC walls with lower aspect ratios is observed to be more significant than that of RC walls with higher aspect ratios. For RC walls with an aspect ratio of 0.5, the secant stiffness significantly reduces from 584.4 kN/mm to 136 kN/mm (around 300% degradation) with the increase of wall drift ratios from 0.1% to 0.5%. While for RC walls with aspect ratios of 1.125, 1.625 and 2.0, this decrease rate is obtained to be 166%, 177%

and 180%, respectively.

5.3.3.4.3 Effect of aspect ratio on energy dissipation

Fig. 5.42 displays the variation of energy dissipation capacity with the drift ratios for RC walls under four aspect ratios. In general, the energy dissipation capacity increases with the added aspect ratios of RC walls studied at same drift ratios. However, up to a drift ratio up to 0.5%, the increase of energy dissipated for all RC walls with aspect ratios varying from 0.5 to 2.0 is observed to be insignificant, while for wall drift ratios larger than 0.5% this increase of energy dissipated becomes obvious. With the augment of aspect ratios from 1.125 to 2.0, the increase rate of dissipated energy is obtained to be 11.6% corresponds to a wall drift ratio of 0.5% while this increase rate is 36.1% for RC walls at a drift ratio of 1.0%.

5.3.3.4.4 Effect of aspect ratio on equivalent damping

The effect of aspect ratios on equivalent damping of walls studied is shown in Fig. 5.43. In general, the equivalent damping increases with higher aspect ratios of RC walls at the same drift ratios. However, with an increase of wall drift ratios, the variation of equivalent damping is observed to be different for all RC walls studied. At low drifts of up to a ratio of 0.25% (prior to reinforcement yielding), analytical equivalent damping for RC walls with four different aspect ratios, decreases significantly from maximum (8.95%) to minimum equivalent damping (2.96%) with the drift ratios varying from 0.1% to 0.25%. Whereas after that, the equivalent damping is observed to increase significantly till the final stage for all of the RC walls studied. At an aspect ratio of

1.125, the percentage increase of equivalent damping is obtained to be 377%, while for RC walls with an aspect ratio of 2.0, this increase rate is 402% with the augment of drift ratios from 0.33% to 1.0%.

5.3.3.5 Effect of Construction Joints

The effect of construction joints on structural performances is investigated for both RC structural walls with and without axial loads, and analytical results of these RC walls with construction joints at the wall base are compared to those of RC walls without construction joints. The poorly detailed construction joints are simulated by nonlinear interface models as mentioned above and are incorporated to the base of the finite element wall model. The tested specimen, Specimen LW1 with an aspect ratio of 1.125, is selected as the reference wall. The overall dimensions and reinforcement details of the studied walls with construction joints are kept same as the reference one.

5.3.3.5.1 Effect of construction joints on wall strength

The effect of construction joints on backbone curves of load-displacement relationships of RC walls studied is presented in Fig. 5.44. It is shown from Fig. 5.44 that the strength capacity is approximately the same, but strength degradation increases slightly for specimens with construction joints at the base, although its ductility level remains almost unchanged.

5.3.3.5.2 Effect of construction joints on secant stiffness

Fig. 5.45 shows the effect of construction joints on secant stiffness of RC structural walls studied. The existence of construction joints has a minor effect on the secant stiffness of RC walls studied since similar values of secant stiffness are observed for walls with or without construction joints.

5.3.3.5.3 Effect of construction joints on energy dissipation

Fig. 5.46 demonstrates the effect of construction joints on energy dissipation of RC walls studied. As indicated in Fig. 5.46, almost similar wall strain energy is dissipated by walls with and without construction joints at a low drift ratio. However, with the increase of wall drift ratios, it is observed that RC walls without construction joints dissipate more strain energy than those with construction joints. This can be due to the fact that with the presence of construction joints at the wall base, sliding deformation along the construction joints may occur and thus the degree of the pinching of load-displacement loops is increased which reduces the energy dissipation capacity of RC walls.

5.3.3.5.4 Effect of construction joints on equivalent damping

The effect of construction joints on equivalent damping of walls studied is presented in Fig. 5.47. In general similar analytical equivalent damping ratios are observed for RC walls with or without construction joints. This indicates that the effect of construction joints at the wall base on the wall equivalent damping is negligible.

5.4 Conclusions

This study develops a nonlinear finite element procedure to explore the structural performance of RC structural walls under cyclic loadings. Several conclusions derived from this study can be made as follows:

1. After calibration against experimental results, the developed nonlinear finite element procedure incorporating realistic material constitutive relations can satisfactorily predict both global and local responses of tested RC structural walls under cyclic loadings.
2. The investigations of the effect of four axial load ratios from 0.00 to 0.15 on the structural performance of RC walls studied found that the presence of axial loads significantly increases the wall strength but the rate of increase becomes less when the RC walls subjected to a higher level of axial loads (from 0.10 to 0.15 etc). Moreover the contribution of axial load to the wall strength for RC walls with a lower aspect ratio is found to be more significant than those with higher aspect ratios. The secant stiffness of walls at the same drift ratio increases with the added axial load, however, this effect reduces with the increase of top drift. The presence of axial loads plays a beneficial effect on the wall energy dissipation capacity at a high drift ratio, while at a low drift ratio this effect is rather negligible. The equivalent damping of RC walls studied decreases with the added axial load when the test progressed. At a low drift ratio, a lower equivalent damping is obtained for RC walls under axial load ratio of 0.05, whereas at a high drift ratio the equivalent damping is observed to

be higher than those of walls without axial load.

3. With the added longitudinal reinforcement content in boundary elements from 0.7% to 4.2%, the analytical maximum strength increases almost linearly and the percentage increase of the maximum strength is observed to be 140% around. The contribution of longitudinal reinforcement to the wall secant stiffness is more effective for walls at higher drift ratios as the rate of increase of secant stiffness increases more rapidly after the attainment of a wall drift ratio of 0.33% for all four RC walls studied. At a high drift ratio (from 0.5% to 1.0%), the increase of longitudinal reinforcement in boundary elements have a significant effect on the wall energy dissipation capacity as the rate of increase of energy dissipation for RC walls at a drift ratio of 1.0% is obtained to be 51% With the added longitudinal reinforcement contents from 0.7% to 4.2%. While at a low drift ratio (up to 0.33%) this effect is rather negligible as for wall drifts of up to a ratio of 0.33%, the energy dissipation almost remains to be same. In general, the equivalent damping for RC walls studied decreases with the added longitudinal reinforcements.
4. The wall strength increases significantly with the increase of boundary column area ratios, A_c / A_t from 0.15 to 0.60. Moreover, the trend of strength increases is observed to be more significant as column area ratios tend to be higher. With the change of area ratios from 0.15 to 0.30, the analytical maximum strength increases relatively to be 0.11, while the increasing values

are obtained to be 0.21 and 0.63 corresponding to the area ratios ranging from 0.30 to 0.45 and 0.45 to 0.60, respectively. The secant stiffness of RC walls with same drift ratios increases with the added column area ratios from 0.15 to 0.60 as it increases by about 58% for RC walls at a drift ratio of 0.1%. However, the rate of increase of secant stiffness with the augment of column area ratios, decreases more rapidly after a wall drift ratio of 0.10% for all four studied RC walls. In general, the energy dissipation capacity and equivalent damping increases with the added area ratios of boundary columns. However, at a high area ratio, the increase of area ratios of boundary columns have a significant effect on the wall energy dissipation capacity and equivalent damping, while at low area ratios (from 0.15 to 0.30), this effect is rather negligible.

5. The wall strength capacity is observed to decrease significantly with the augment of wall aspect ratios as the percentage decrease of the analytical maximum strength is observed to be 47.7%, 63.3% and 70.7% with the increase of aspect ratios from 0.50 to 1.125, 1.625 and 2.0, respectively. With the augment of wall drift ratios, the rate of decrease of the secant stiffness for RC walls with lower aspect ratios is observed to be more significant than that of RC walls with higher aspect ratios. In general, the energy dissipation capacity and equivalent damping increases with higher aspect ratios of RC walls studied at same drift ratios. However, up to a drift ratio up to 0.5%, the increase of energy dissipated for all RC walls with aspect ratios varying from

0.5 to 2.0 is observed to be insignificant, while for wall drift ratios larger than 0.5% this increase of energy dissipated becomes obvious.

6. The existence of construction joints has a minor effect on the secant stiffness of RC walls studied since similar values of strength and secant stiffness are observed for walls with or without construction joints. Almost similar wall energy is dissipated by walls with or without construction joints at a low drift ratio. However, with the increase of wall drift ratios, walls without construction joints dissipate more strain energy than those with construction joints. Similar analytical equivalent damping ratios are obtained for walls without or with construction joints.

Table 5.1 – Coefficients for determination of the fracture energy

$d_{\max}$ (mm)	8	16	32
G_{f0} (N · mm / mm ²)	0.025	0.030	0.058

Table 5.2 - Material properties and reinforcement ratio for
Unit 1.0 and Specimen S-F1

Properties		Unit 1.0	Specimen S-F1
Cross section shape		Barbell	Barbell
Concrete compressive strength	(N/mm ²)	25.0	35.0
Concrete fracture energy	(Nmm/mm ²)	0.051	0.072
Concrete compressive fracture	(Nmm/mm ²)	7.65	10.8
Yield strength (N/mm ²)	Boundary element	492.0	385.0
	Vertical web	298.0	492.0
	Horizontal web	298.0	492.0
Reinforcement ratio, ρ (%) at	Boundary (Long.)	3.00	1.30
	Vertical web	0.16	0.26
	Horizontal web	0.16	0.24

Table 5.3 – Comparisons of finite element predictions to test results

Specimen	Tested		Predicted		Tested/Predicted	
	$F_{p,t}$ (kN)	$\Delta_{p,t}$ (mm)	$F_{p,p}$ (kN)	$\Delta_{p,p}$ (mm)	$F_{p,t}/F_{p,p}$	$\Delta_{p,t}/\Delta_{p,p}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Unit 1.0	483.2	17.75	504.6	18.50	0.96	0.96
S-F1	393.3	20.19	355.0	20.00	1.11	1.01

Table 5.4 – Concrete material properties used for finite element analysis

Specimens Properties	LW1	LW2	LW3	LW4	LW5	MW1	MW2	MW3
f'_c (N/mm^2)	40.2	41.6	34.8	39.8	35.6	41.2	39.6	40.3
f_t (N/mm^2)	3.36	3.42	3.13	3.34	3.16	3.40	3.33	3.36
E_c (N/mm^2)	34185	34578	32581	34072	32829	34467	34015	34214
G_f ($N \cdot mm/mm^2$)	0.083	0.086	0.071	0.082	0.073	0.085	0.081	0.083
G_c ($N \cdot mm/mm^2$)	12.4	12.8	10.7	12.3	10.9	12.7	12.2	12.4

Table 5.5 - Parameters investigated

No.	Name	Description	Range Investigated
1	$N/(f'_c A_g)$	Axial load ratio	0.00, 0.05, 0.10, 0.15
2	ρ_b (%)	Longitudinal reinforcement content in boundary element	0.70, 1.40, 2.80, 4.20
3	A_c / A_t	Ratio of the sectional area of columns to the total sectional area	0.15, 0.30, 0.45, 0.60
4	h_w / l_w	Aspect ratio	0.5, 1.125, 1.625, 2.0
5	CJ	Construction joints	Cold Joints, No joints

Fig. 5.1 – Concrete in compression and tension

Fig. 5.2 - Shear friction hypothesis [D1]

Fig. 5.3 - Coulomb friction criterion

Fig. 5.4 – Aggregate interlock relation [W1]

Fig. 5.5 – The overall dimensions and reinforcement layout of Unit 1.0 [M1]

Fig. 5.7 – Finite element idealization and deformed shapes of Unit 1.0 at a drift ratio of 1.0%

Fig. 5.6 – The overall dimensions and reinforcement layout of Specimen S-F1 [W2]

Fig. 5.8 – Finite element idealization and deformed shapes of Specimen S-F1 at a drift ratio of 1.0%

Fig. 5.9 – Experimental and analytical hysteretic responses of Unit 1.0

Fig. 5.10 – Experimental and analytical hysteretic responses of Specimen S-F1

Fig. 5.11 – Finite element idealization for all specimens tested

(a)

(b)

(c)

(d)

(e)

(f)

Fig. 5.12 – Experimental and analytical hysteretic responses of all specimens tested

(a) Section 1-1 of Specimen LW1

(b) Section 2-2 of Specimen LW1

(c) Section 1-1 of Specimen LW2

(d) Section 2-2 of Specimen LW2

Fig. 5.13 – Verification of longitudinal strain distribution along wall length

(c) R bar in Specimen LW2

(d) T bar in Specimen LW2

(e) R bar in Specimen MW1

(f) T bar in Specimen MW1

Fig. 5.14 – Verification of horizontal strain distribution in selected horizontal bars

Fig. 5.15 – Representation of the secant stiffness, energy dissipation capacity and equivalent damping factors

Fig. 5.16 – Effect of axial load on backbone curves of load-displacement loops

Fig. 5.17 – Contribution of axial load ratio to the wall strength

Fig. 5.18 – Effect of axial load on secant stiffness of walls studied

Fig. 5.19 – Contribution of axial load ratio to the wall secant stiffness

Fig. 5.20 – Effect of axial load on energy dissipation of walls studied

Fig. 5.21 – Contribution of axial load ratio to the energy dissipation

Fig. 5.22 – Effect of axial load on equivalent damping of walls studied

Fig. 5.23 – Contribution of axial load ratio to the equivalent damping

Fig. 5.24 – Effect of longitudinal reinforcement content on backbone curves of load-displacement loops

Fig. 5.25 – Contribution of longitudinal reinforcement content to the wall strength

Fig. 5.26 – Effect of longitudinal reinforcement content on secant stiffness

Fig. 5.27 – Contribution of longitudinal reinforcement content to secant stiffness

Fig. 5.28 – Effect of longitudinal reinforcement content on energy dissipation

Fig. 5.29 – Contribution of longitudinal reinforcement content to the energy dissipation

Fig. 5.30 – Effect of longitudinal reinforcement content on equivalent damping

Fig. 5.31 – Contribution of longitudinal reinforcement content to equivalent damping

Fig. 5.32 – Effect of boundary columns on backbone curves of load-displacement loops

Fig. 5.33 – Contribution of boundary columns to the wall strength

Fig. 5.34 – Effect of boundary columns on secant stiffness

Fig. 5.35 – Contribution of boundary columns to secant stiffness

Fig. 5.36 – Effect of boundary columns on energy dissipation

Fig. 5.37 – Contribution of boundary columns to the energy dissipation

Fig. 5.38 – Effect of boundary columns on equivalent damping

Fig. 5.39 – Contribution of boundary columns to equivalent damping

Fig. 5.40 – Effect of aspect ratio on backbone curves of load-displacement loops

Fig. 5.41 – Effect of aspect ratio on secant stiffness

Fig. 5.42 – Effect of aspect ratio on energy dissipation

Fig. 5.43 – Effect of aspect ratio on equivalent damping

Fig. 5.44 – Effect of construction joints on backbone curves of load-displacement loops

Fig. 5.45 – Effect of construction joints on secant stiffness

Fig. 5.46 – Effect of construction joints on energy dissipation

Fig. 5.47 – Effect of construction joints on equivalent damping

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NOTATIONS

A_h	Energy dissipation capacity
$A_{h,i}$	Energy dissipation at the i th level of parameters investigated
$A_{h,0}$	Energy dissipation at initial level of parameters investigated
A_g	Gross cross section area
c	Cohesion
D	Tangential stiffness matrix
f_c'	Cylinder strength of concrete
F_m	Peak lateral load
$F_{p,p}$	Predicted peak loads
$F_{p,t}$	Tested peak loads
f_t	Concrete tensile strength
G_c	Compressive fracture energy
G_f	Tensile fracture energy
h	Crack band width
h_{eq}	Equivalent damping factor
$h_{eq,i}$	Equivalent damping at the i^{th} level of parameters investigated
$h_{eq,0}$	Equivalent damping at initial level of parameters investigated
K_{eq}	Secant stiffness
$K_{i,a}$	Analytical secant stiffness at the i^{th} level of parameters investigated

$K_{0,a}$	Analytical secant stiffness at initial level of parameters investigated
N	Axial load
P	Clamping force perpendicular to the sliding plane
$P_{i,max}$	Maximum strength at the i^{th} level of parameters investigated
$P_{0,max}$	Maximum strength at initial level of parameters investigated
ρ_b	Flexure reinforcement ratio in boundary element
$\epsilon_{nn,ult}^{cr}$	Ultimate crack strain
t	Force traction vector
t_n	Tensile traction
μ	Friction coefficient
V	Shear force
δ	Separation between the two halves
ψ	Dilatancy angle
ϕ	Friction angle
Δu	Relative displacements vector
Δ_m	Peak lateral displacement
$\Delta_{p,p}$	Predicted associated displacements
$\Delta_{p,t}$	Tested associated displacements

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

An experimental program is carried out to explore the seismic performance of eight squat RC structural walls with limited transverse reinforcement. The local and global responses of the RC structural walls tested under cyclic loadings are described in detail. The influence of several design parameters, such as axial loads, transverse reinforcements in the wall boundary columns and the presence of construction joints at the wall base, on the local and global responses of these RC structural walls are also reported in this experimental program. Based on the observed experimental results, reasonable strut-and-tie models for squat RC structural walls with and without axial loads are developed to aid in better understanding the force transfer mechanism and contribution of reinforcement in RC walls tested.

Next, an analytical procedure incorporating both shear and flexure deformations, is presented to properly evaluate the initial stiffness for RC structural walls with low aspect ratios. A comprehensive parametric study including a total of 180 combinations is carried out and a simple expression is proposed to determine the wall initial stiffness as a function of three factors: yield strength of the outermost longitudinal reinforcement, applied axial compression and wall aspect ratios. The proposed stiffness formulae are validated with experimental results and its effectiveness in stiffness predictions are

further found by comparison with previous stiffness formulas.

Finally, a nonlinear finite element analytical procedure is used in this study to investigate the global responses of squat RC structural walls under cyclic loadings. By employing this nonlinear analytical procedure, the global responses such as strength capacity, stiffness characteristics, energy dissipation capacity and equivalent damping of these squat RC structural walls under reversed seismic loadings are described in detail and calibrated against the experimental ones. Furthermore, based on the verified analytical procedure, an extensive parametric study is carried out to investigate the influence of several design parameters such as axial loads, longitudinal reinforcements in the wall boundary elements, aspect ratios, area ratio of boundary columns and the presence of construction joints at the wall base, on the global responses of squat RC structural walls.

Several conclusions, as listed below, can be drawn based on these experimental and analytical investigations:

1. In general, all eight tested specimens with limited transverse reinforcement behave in a flexural manner and are capable of developing their flexural capacity prior to failure. Values of drift at initial cracking range from 0.1% to 0.17% and 0.17% to 0.25% for specimens with aspect ratios of 1.125 and 1.625, respectively.
2. Specimens with an aspect ratio of 1.125, specimens LW1, LW2, LW4 and LW5, generally exhibit more ductile behavior than expected, even if they have insufficient confinement reinforcement corresponding to 70% and 25% of the

NZS 3101 and ACI 318 specified quantity of confining reinforcement, respectively. The displacement ductility factors of the four specimens are more than 3.0 and can generally experience an average story drift of at least 1% without significant strength degradation. By contrast, Specimen LW3, containing 30% and 10% of the NZS 3101 and ACI 318 specified quantity of confining reinforcement respectively, shows quite critical seismic performance with respect to the strength and deformation capacities achieved. The displacement ductility of Specimen LW3 is observed to be less than 3.0, which proves to exhibit only limited ductile behavior of the wall as stipulated in NZS 3101 code.

3. For specimens with an aspect ratio of 1.625, the content of transverse reinforcements at the wall boundaries, a reduction to 30% and 10% that is required by NZS 3101 and ACI-318 code corresponding to fully ductile walls, might be considered as an effective measure for confining the concrete in the compression zone in terms of the limited ductile performance of walls.
4. Moreover, for the specimens with more content of the transverse reinforcements in wall boundaries, the flexural contribution of the total deformation increases while the shear component of total displacements decreases. This indicates clearly that seismic performance such as drift, ductility and energy dissipation capacity can be enhanced by increasing the amount of the transverse reinforcement at the boundary elements of a wall. However, it is also found that the content of transverse reinforcement in wall boundary element and the presence of construction joints at the wall base have negligible effects on the stiffness characteristics of the RC walls tested.

5. The tests showed that the peak load attained for structural walls with construction joints are almost the same as those without construction joints and both walls can sustain the ultimate loads well into the post-elastic range. However, the drop off in load for structural walls with construction joints, after the peak had been attained, was larger than that for structural walls without construction joints. This indicates that the stiffness degradation for structural walls with joints, especially for structural walls with an aspect ratio of 1.125, is large after the peak load has been attained. However, in terms of limited ductile criteria, structural performances such as ductility, strength and energy dissipation capacities for tested walls with construction joints at the base can still be considered to be adequate. Therefore, the usual object that the surrounding concrete acts as a monolithic part of the member can be achieved in the design of squat RC walls with a construction joint.
6. From the testing, it also can be concluded that the level of axial compression has a minor effect on the degradation rate of secant stiffness despite the fact that the presence of axial compressions in specimens can lead to higher secant stiffness in contrast with those without axial loads. For the wall specimens subjected to axial compression, the amount of energy dissipated is larger than that corresponding to specimens without axial loads due to the favorable effect of the axial compression with regard to controlling the pinching of hysteresis loops.
7. By decomposing the total lateral deformation into flexural and shear components as well as sliding components, it can be demonstrated that the bulk of the energy dissipation is due to flexure. The amount of energy dissipated due

to shear components does not change much under the condition of axial loadings on the specimen. With regards to the energy dissipation contributed by sliding components, it is found to increase slightly due to the presence of construction joints at the wall base, but still remained at a low level up to the final stage of testing.

8. The developed strut-and-tie analytical models for specimens tested with and without axial load respectively, accounting for different contributions of horizontal and longitudinal web reinforcement, are found to be able to accurately reflect the force transfer mechanisms of RC structural walls under cyclic loadings. Moreover, the analytical tensile strains in horizontal web bars of RC structural walls predicted by use of the developed strut-and-tie models agree well with the tested data. This provides evidence that the assumed strut-and-tie models are reasonable for the flow of forces and contribution of web reinforcements in walls tested.
9. By comparison with the results from the current tested results, it is found that the developed analytical approach, incorporating both the flexure and shear deformations, can effectively evaluate the initial stiffness of RC low-rise structural walls with limited transverse reinforcement.
10. Analysis of the parametric study, shows that three critical parameters: yield strength of outermost longitudinal reinforcement, applied axial load, and wall aspect ratios, influence the initial stiffness of low-rise structural walls most. Based on this parametric study, a simple expression to evaluate the wall initial stiffness accounting for both flexure and shear deformations is proposed and

validated with the tested data. The results obtained are in good agreement with the experimental work.

11. By applying the concrete constitutive law as a total strain rotating crack model and a reinforcing bar as the Von Mises plastic material considering the strain hardening rule, a nonlinear finite element analysis procedure is used to extensively explore structural performance of squat RC structural walls under cyclic loadings. Based on regular Newton-Raphson iteration method, this developed analytical procedure is found to be able to satisfactorily predict both global and local responses of tested RC structural walls under cyclic loadings.
12. The investigations of the effect of four axial load ratios from 0.00 to 0.15 on the structural performance of squat RC structural walls found that the presence of axial loads significantly increases the wall strength but the rate of increase becomes less when the RC walls subjected to a higher level of axial loads (from 0.10 to 0.15 etc). Moreover the contribution of axial loads to the wall strength for RC walls with a lower aspect ratio is found to be more significant than those with a higher aspect ratio. The secant stiffness of walls at the same drift ratio increases with the added axial load, however, this effect reduces with the increase of the top wall drift. The presence of axial loads on RC walls is found to play a beneficial effect on the wall energy dissipation capacity at a high drift ratio, while at a low drift ratio this effect is negligible. The equivalent damping of RC walls studied decreases with the added axial load when the test progressed. At a low drift ratio, a lower equivalent damping is obtained for RC walls under axial load ratio of 0.05, whereas at a high drift ratio the equivalent damping is observed to be higher than that of walls without axial load.

13. With the addition of longitudinal reinforcement content in boundary elements from 0.7% to 4.2%, the analytical maximum strength increases almost linearly. The contribution of longitudinal reinforcements to the wall secant stiffness increases more rapidly after the attainment of a wall drift ratio of 0.33%. The increase of longitudinal reinforcement in boundary elements have a more significant effect on the energy dissipation capacity for RC walls at a high drift ratio (from 0.5% to 1.0%) than for those up to a drift ratio of 0.33% as its energy dissipation almost remains the same. In general, the equivalent damping for RC walls studied decreases with the addition of longitudinal reinforcements in boundary elements.
14. The wall strength increases significantly with the augmentation of boundary column area ratios, A_c / A_t from 0.15 to 0.60 and the rate of increases is observed to be higher as column area ratios tend to be larger. The secant stiffness of RC walls increases with the added column area ratios, however, the rate of increase of secant stiffness with the augmentation of column area ratios, decreases rapidly after a wall drift ratio of 0.10%. In general, the energy dissipation capacity and equivalent damping increases with the addition of area ratios of boundary columns and the rate of increase becomes more significant as area ratios of boundary columns tend to be larger.
15. The wall strength capacity is observed to decrease significantly with the augmentation of wall aspect ratios as the percentage decrease of the analytical maximum strength is observed to be 47.7%, 63.3% and 70.7% with the increase of aspect ratios from 0.50 to 1.125, 1.625 and 2.0, respectively. With the augment of wall drift ratios, the rate of decrease of the secant stiffness for RC

walls with lower aspect ratios is observed to be more significant than those of RC walls with higher aspect ratios. In general, the energy dissipation capacity and equivalent damping increases with the addition of aspect ratios of RC walls at same drift ratios. However, up to a drift ratio of 0.5%, the increase of energy dissipated for all RC walls with aspect ratios varying from 0.5 to 2.0 is observed to be negligible, while for wall drift ratios larger than 0.5% this increase of energy dissipated becomes significant.

16. The structural interface model based on the shear friction theory is adopted for RC structural walls with construction joints at the base. The analytical results using this model correspond well with the experimental ones and this indicates that this shear friction model is suitable for simulation of joint behaviors if a suitable friction coefficient for the surface is chosen. The existence of construction joints has a minor effect on the secant stiffness of walls studied since similar values of strength and secant stiffness are observed for walls with or without construction joints. Also, almost similar wall energy is dissipated by walls with or without construction joints at a low drift ratio. However, with the increase of wall drift ratios, walls without construction joints dissipate more strain energy than walls with construction joints. Similar analytical equivalent damping ratios are obtained for both squat RC walls with and without construction joints.

6.2 Recommendations for Future Works

Several issues related to this study deserve further investigation. This includes:

1. Experimental programs should be carried out to further investigate the effect of boundary column areas on the seismic performance of squat RC structural walls with limited transverse reinforcement.
2. The analytical initial stiffness predicted by the proposed nonlinear finite element procedure is normally larger than that from testing. This may due to the perfect bond assumed between the longitudinal reinforcement and associated concrete. Accordingly in the future analytical works a reasonable structural interface model between the reinforcement and concrete should be developed and incorporated to the finite element analytical model.
3. More experimental stiffness data for squat RC structural walls under cyclic loadings are needed to calibrate the effectiveness of the proposed stiffness formulas.
4. In current nonlinear finite element analysis of squat RC walls, the structural interface model simulating the construction joints is based on the shear friction analogy, and parameters such as normal and shear stiffness of the interface affecting the models are mostly shown to be semi-empirical. Moreover, the friction coefficient in the model is assumed to be constant throughout the analysis, which does not represent the true behavior of the interface. In fact, the true behavior of the interface including aggregate interlock of the surface along the construction joints and dowel action of the reinforcement bars crossing this surface will change significantly as the test progressed and has not been explicitly represented in this finite element analysis due to its complexity. Accordingly, extensive research should be carried out to investigate the true

behavior of the interface along the construction joints when the test progressed and more precise finite element models involving the aggregate interlock of the interface and dowel action of reinforcement bars crossing the interface should be incorporated to the future work.

5. By employing the proposed finite element analytical procedure, the local responses of squat RC structural walls under cyclic loadings should be explored further.