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INELASTIC BEHAVIOUR OF REINFORCED MASONRY SHEAR WALLS: AN EXPERIMENTAL AND ANALYTICAL STUDY

By

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A thesis submitted to
the Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of

DOCTOR OF PHILOSOPHY

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CARLETON UNIVERSITY
OTTAWA, CANADA**

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**Inelastic Behaviour of Reinforced Masonry Shear Walls:
An Experimental and Analytical Study**

submitted by

Khaled S. Ibrahim, B.Sc., M.Eng.

in partial fulfillment of the requirements for
the degree of Doctor of Philosophy

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Abstract

In multi-storey masonry buildings, reinforced masonry shear walls usually form the lateral seismic load resistance elements. The design of reinforced masonry shear walls for all critical seismic limit states requires a detailed understanding of the complete reversed cyclic response. Therefore, large scale experimental data are necessary to establish behavioural characteristics of masonry shear walls subjected to cyclic loading before comprehensive and representative design models can be developed. Observing the physical characteristics of masonry shear walls under the aforementioned complex states of stress may lead to a better understanding of the overall behaviour of masonry buildings and may be used to verify existing theoretical models. These models then can be employed to perform a large-scale three dimensional computer simulation utilizing state-of-the-art computing facilities to predict the behaviour of reinforced masonry structures under monotonic and cyclic/seismic loading patterns up to the ultimate stage.

The main objective of this thesis work was to develop a smeared crack finite element based model for reinforced masonry and to evaluate its capabilities in capturing various failure mechanisms of reinforced masonry shear walls undergoing different monotonic and cyclic loading patterns. The accuracy of this model was verified against available experimental evidence as well as against data from a concurrent experimental program which involved the testing of five reinforced fully grouted concrete masonry shear walls. In the experimental program, the influence of three parameters on the performance of reinforced masonry shear walls subjected to cyclic loading was investigated; these three parameters were the wall aspect ratio (wall height/length ratio), the amount of vertical reinforcement and the level of axial compressive stress. Two main types of failure were dealt with analytically and experimentally; these were the relatively ductile flexural failure and the relatively brittle shear failure.

For flexure dominated walls, the analytical load-displacement history matched well the experimental envelope curve. The vertical steel yielding pattern due to the widening of flexural cracks and the extent of the crushing were very similar to that observed experimentally. Close prediction for the wall maximum load resistance was achieved.

For shear dominated walls, the analytical load-displacement history also matched well the experimental envelope curve. The maximum lateral load resistance predicted by the analytical model was slightly higher than that shown in tests and occurred at a somewhat lower lateral displacement. Numerical results using the current model indicated no spurious shear strength for the shear dominated walls and the sudden load decrease in resistance due to the excessive opening of the diagonal crack was properly captured.

Experimental results indicated that a linear relationship can be assumed between wall aspect ratio and lateral load resistances for walls having the same reinforcement ratios and subjected to the same vertical stress level. Even with increased length and increased axial vertical stress, the walls experienced their first major diagonal cracks at almost the same lateral displacements. Also, experimental findings indicated that while increasing the vertical reinforcement ratio has a slight effect on the wall lateral load resistance, increasing the amount of vertical axial load increases the wall lateral load resistance.

Based on the available experimental results, a new formula is proposed for the residual shear strength of masonry shear walls. The proposed formula appears to be more reliable and consistent with the experimental results and recognizes the contribution of the vertical reinforcement, the axial load and the wall aspect ratio on the resistance of masonry shear walls.

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NOTATION

a_{b-1} and a_g	factors representing the fraction of sectional area of block cross webs and grout
A	net or gross horizontal area of the wall
A_h	area of a horizontal reinforcing bar having a nominal yield stress f_{yh}
A_n	net horizontal cross-sectional area
A_{si}	cross-sectional area of a vertical bar i
b	width of the cross-section of the wall
c	distance of the neutral axis from the extreme compressive fiber
c	reduction factor
C_t	empirical constant that has a unit of length
c_1 and c_2	empirical constants
d	effective depth
d_a	maximum aggregate size.
d_b	diameter of reinforcement bars
d'	distance of the vertical tension reinforcement to the nearest jamb
d_i	distance of a bar from the compression edge
D'_m	tangent constitutive matrix of grouted masonry
D'_s	tangent constitutive matrix of smeared reinforcement
D'_s	tangent constitutive matrix of smeared reinforcement
E_t	initial tangent modulus
E_{bm}	initial elastic modulus of block material
E_n, E_s	tangent moduli of steel in the n, s directions
E_g	initial elastic modulus of grout
f_{um}	uniaxial ultimate masonry compressive strain
f_m	masonry stress at any given strain of ϵ_m

f'_m	uniaxial masonry peak compressive stress
$\tilde{f}'_m$	uniaxial masonry peak stress accounting for multiaxial stress conditions
$\tilde{f}_{um}$	uniaxial masonry ultimate stress accounting for multiaxial stress conditions
$\tilde{f}'_m$	uniaxial masonry peak stress accounting for multiaxial stress conditions
f_{yh}	nominal yield stress of the horizontal reinforcing steel
f_{yv}	nominal yield stress of the vertical reinforcing steel
$f(\phi)_{mcr}$	tensile failure stress is related to the angle ϕ , measured from the vertical axis f_{mcr} cracking stress for uniaxial tension normal to the bedjoint and ϕ is in degrees
G_{orth}	torsional rigidity of orthotropic masonry
h_w	wall height
k_1 and k_2	parameters to account for the effect of confinement
k	constant
k_o and k_w	dimensionless factors depend on the masonry type
l_w	wall length
L_g	spacing of grouted cores
M	factored moment at the section under consideration
n	number of reinforcing bars
n_c	number of cross webs per unit length of the wall
P	axial compressive load on the section under consideration
r	aspect ratio of a wall
s	spacing of horizontal reinforcement
t_g	thickness of grouted cores
t_w	thickness of the cross webs of blocks
T	transformation matrix that is the function of the angle between the global coordinate axes and the axes of orthotropy β
v_a	contribution of axial compressive load to shear strength

v_n	nominal shear stress of masonry at ultimate
v_m	contribution of masonry to shear strength
v_s	contribution of horizontal reinforcement to shear strength
V	factored shear at the section under consideration
V_n	nominal ultimate shear strength
V_m	residual strength of masonry after diagonal cracking
V_r	in-plane shear resistance of a masonry wall
V_s	shear resistance of the horizontal reinforcement
V_u	total shear strength of a masonry wall
w	crack width
w_c	crack band width
α	strain-hardening parameter for steel
β	empirical constant to account for the reduction in the masonry compressive strength due to the tensile principal strain in the direction normal to that being considered
γ_1	stress factor to account for multiaxial stress conditions
γ	strain factor to account for multiaxial stress conditions
γ and β	the stress block parameters
γ_g	a factor to account for partially grouted walls
ϵ_o	strain at uniaxial peak compressive stress
ϵ_{um}	uniaxial ultimate masonry compressive stress
$\tilde{\epsilon}_{um}$	strain at uniaxial masonry ultimate stress $\tilde{f}_{um}$
$\tilde{\epsilon}_o$	strain at uniaxial masonry peak stress $\tilde{f}_o$
ϵ_{m1}	the tensile principal strain in the direction normal to that being considered
ϵ_{m1}^c	absolute tensile principal strain in the direction normal to that being considered
ϵ_{m1}^c	absolute masonry tensile principal strain under cyclic loading condition in the direction normal to that being considered
ϵ_{mcr}	value of strain at f_{mcr}

η	constant (between 1.5 and 4)
ϵ_{un}	unloading masonry tensile strain
ϵ_r	residual masonry tensile strain
λ	constant
κ	variable defines the state of isotropy for masonry
ν	Poisson's ratio
ψ_m	resistance reduction factor for masonry
ϕ_s	resistance reduction factor for reinforcing steel
ρ_h	ratio of the horizontal steel with respect to the vertical section through the wall
ρ_v	ratio of vertical reinforcement
ρ_n and ρ_s	reinforcement ratios in the n and s directions
ρ_s	reinforcement percentage
σ_c	axial compressive stress
τ_u	shear strength of a masonry wall
τ_m	residual shear strength of a masonry wall

Chapter 1

INTRODUCTION

1.1 General

Analytical and experimental investigations of components, subassemblages, and prototypes of reinforced masonry buildings are used to provide a better understanding of the principal behavioural characteristics of masonry structures and to help in deriving rational design models and codes. Costs and laboratory facilities impose significant limitations on the extent of full-scale testing. Therefore, current research efforts are mainly concentrated on studying experimentally the physical behaviour of masonry components and utilizing the experimental results to verify and validate up-to-date simple or complex analytical models.

Lateral motion will result when buildings are subjected to lateral ground shaking due to passage of seismic waves. This motion will induce additional forces

and stresses on the lateral load-resisting structural elements. In multi-storey masonry buildings, reinforced masonry shear walls usually form the lateral seismic load resistance elements. The design of reinforced masonry shear walls for all critical seismic limit states requires a detailed understanding of the complete reversed cyclic response. Therefore, large scale experimental data are necessary to establish behavioural characteristics of masonry shear walls subjected to cyclic loading before comprehensive and representative design models can be developed. Observing the physical characteristics of masonry shear walls under the aforementioned complex states of stress may lead to a better understanding of the overall behaviour of masonry buildings and may serve to verify existing theoretical models. These models in turn can be employed to perform a large-scale three-dimensional computer simulation utilizing state-of-the-art computing facilities to predict the behaviour of reinforced masonry structures under monotonic and cyclic/seismic loading patterns up to the ultimate stage.

The aforementioned problem is highly nonlinear in nature and various models have been developed to trace this behaviour. The finite element method offers a powerful tool for modeling complex structural systems under various loading patterns and tracing the complete nonlinear behaviour up to the ultimate stage. Using this technique, various physical characteristics of reinforced masonry such as post-cracking or post-crushing phenomena can be modeled successfully.

Ngo and Scordelis (Ngo and Scordelis, 1967) in the early times of finite element analysis modeled cracks by means of a separation between element nodes using special spring elements where they were connected. A large dummy value is assigned for the initial stiffness of the elements to simulate the uncracked state with rigid connection between overlapping nodes. Upon violating a condition of crack initiation, the element stiffness is changed and a constitutive model for discrete cracks is utilized to simulate the physical behaviour of the interface. This approach has different drawbacks. The crack is constrained to follow a predefined path along the element edges where special interface elements exist and their location is known

in advance; also the physical characteristics of the joints are difficult to obtain experimentally.

A continuous interface element was developed by Goodman *et al.* (Goodman, Taylor, and Brekke, 1968) to model the interface behaviour through an interpolated field. Using the original node displacement as independent degree-of-freedom may cause an ill-conditioning state which added another drawback to the discrete crack model. This problem was eliminated using an interface element with relative displacements between both edges of the interface as independent degrees-of-freedom. A state-of-the-art review on interface elements can be found in details elsewhere (Ibrahim, 1989). This approach, which resembles the distinct element method pioneered by Cundall and Strack (Cundall and Strack, 1979), has become very popular recently in fracture mechanics research.

Contrary to the discrete crack model, the smeared crack model imagines the cracked solid as a continuum with new physical properties. This approach was first introduced by Rashid (Rashid, 1968) and can be summarized as switching from the initial isotropic stress-strain law to an orthotropic law upon crack formulation. In this case, the axes of orthotropy are determined according to the crack initiation. Two advantages made this procedure very attractive. Firstly, it preserves the topology of the original finite element mesh. Secondly, it does not constrain the orientation of the crack planes. Therefore, in the early seventies, the smeared crack concept quickly replaced the discrete crack concept and became very widespread in its use.

Much controversy has arisen ever since cracking has been modeled using either discrete or smeared crack models. While the discrete crack model appears to be more reliable where the fracture is identified as a true geometric discontinuity, a smeared crack concept might be more realistic when the bands of micro-cracks, at the tip of a visible crack, that blunt fracture in matrix-aggregate composites are considered (Bazant and Oh, 1983). Since experimental evidence of crack tip related micro-mechanics damage in matrix-aggregate composites is contradictory

and scarce, it is difficult to judge the argument whether the damage process occurs in a discrete manner or not (Diamond and Bentur, 1985, Tait and Garret, 1986). Also, a true physical basis for smeared concepts is given by the diffuse crack patterns in large-scale shear walls due to the presence of densely distributed reinforcement. In this case, the smeared concept seems the only rational approach rather than the unrealistic discrete fracture concept, where every single crack has to be modeled from physical and numeric points of views.

1.2 Scope and Objectives

A smeared crack model approach has been used in finite element analyses of concrete structures. While such a technique proved to be reliable and numerically efficient in many instances, its capability in predicting the mechanical behaviour for a vast variety of matrix-aggregate composites has not been thoroughly studied especially in the field of complex composite materials such as reinforced masonry.

While numerous problems are associated with the smeared crack model such as mesh-size dependency of numerical solution, failure to model elements with dominant shear failure and spurious kinematic modes, it is still the simplest and most efficient approach to model the fracture behavior of reinforced masonry shear walls exhibiting diffuse crack patterns.

The main objectives of the current study are:

- to develop a smeared crack model for reinforced masonry and evaluate its capabilities in capturing various failure mechanisms of reinforced masonry shear walls undergoing different monotonic and cyclic loading patterns;
- to conduct an experimental investigation on various key parameters affecting the lateral resistance of selected reinforced masonry shear walls that will describe the full range of behaviour missing in the published experimental results;
- to verify the finite element predictions with existing experimental evidence to calibrate the analytical model;
- to compare finite element predictions with the response of the test group;

- to study the design implications of the experimental findings;
- to develop rational mathematical models to describe the behaviour of reinforced masonry shear walls subjected to lateral in-plane cyclic loading.

This thesis work divides into two major concurrent phases. First is a theoretical phase, where a smeared-crack finite element formulation adopting a hypoelastic model for uncracked masonry and nonlinear orthotropic constitutive models for cracked masonry have been developed. In this numerical model most of the current knowledge in the field of reinforced masonry modeling, such as fixed, coaxial rotating, non-coaxial rotating crack representation are incorporated. The Compression Field Theory (CFT) and Modified Compression Field Theory (MCFT) have been implemented. Numerous constitutive models for reinforced masonry under tension and compression have been incorporated into the program and their significance on the numerical solution has been assessed. Post-tension cracking and post-compression crushing behaviour have been modeled. Strain-softening behaviour and different tension-stiffening models are incorporated in the numerical solution. Also, the influence of the anisotropy of partially grouted masonry has been considered.

Very few experimental data for single storey plane and flanged shear walls have been reported. The published experimental studies were limited to the investigation of the influence of certain parameters, such as the design and loading conditions, on the lateral shear resistance of reinforced masonry walls. No experimental evidence was available for the effect of the wall geometry on the shear resistance of reinforced masonry shear walls.

Therefore, as a second phase and after reviewing the available published experimental evidence, it was decided to experimentally investigate the influence of the wall aspect ratio (wall height/length ratio), the amount of vertical reinforcement and the level of axial compressive stress on the performance of reinforced masonry shear walls subjected to cyclic loading while undergoing static vertical loading. These results along with the available published experimental data pro-

vided enough information to verify the numerical model developed in the first phase of the thesis work.

1.3 Outline of the Thesis

Chapter 2 presents a literature review of the analytical studies conducted to predict the behaviour of reinforced masonry shear walls and of the experimental programs conducted to study the influence of various parameters including geometric, design and loading conditions on the lateral resistance of reinforced masonry shear walls. Chapter 3 details the finite element based smeared crack model including the finite element procedure employed, constitutive relationships for masonry and steel, failure criteria and numerical incremental-iterative techniques employed in the analytical study. Finally, results were compared with the experimental evidence. A detailed description of the experimental program is reported in Chapter 4 and the test results are presented and discussed in Chapter 5. Comparison between the experimental results and the finite element model predictions are presented in Chapter 6. Design implications and Code related evaluations are presented and discussed in Chapter 7. Conclusions and recommendations from both the analytical and experimental studies are given in Chapter 8.

Chapter 2

LITERATURE REVIEW

2.1 General

While extensive experimental data and analytical models to predict the behaviour of reinforced concrete elements are available in the literature, this is not the case for reinforced masonry. This chapter presents a literature review of the limited analytical models developed or employed for reinforced masonry shear walls. Also included is a literature review of the laboratory experiments carried out on masonry components and sub-assemblages.

2.2 Phenomenological Models for Masonry Structures

Recently, engineered reinforced masonry has proven to be an efficient building material that can tolerate different types of loading and, therefore, has attracted researchers' attention for further studies to fully understand its phenomenological behaviour.

Seible *et al.* (Seible, LaRovere, and Kingsley, 1990) developed a finite element model which can trace the nonlinear behaviour of in-plane masonry building components and substructures. Their phenomenological model was based on higher order isoparametric overlaid elements in which the reinforcement and the fully grouted ma-

sonry were formulated separately and were subsequently connected by compatibility requirements. The constitutive behaviour of the grouted masonry was based on an orthogonally anisotropic material model and smeared crack theory. They claimed close agreement with the experimental results obtained from reinforced masonry panels. Their work was developed as part of the U.S.-TCCMAR research effort to predict the detailed load-deformation characteristics and critical limit states of reinforced masonry in-plane walls.

Lotfi and Shing (Lotfi, and Shing, 1991) developed a smeared-crack finite element formulation adopting the J_2 plasticity model for uncracked concrete and nonlinear orthotropic constitutive models for cracked masonry. The model was only available for the case where the load is applied monotonically. The performance of the model was evaluated using experimental data obtained from testing reinforced masonry panels subjected to lateral loads. It was shown that the model has a major drawback in capturing brittle shear behaviour.

Ewing *et al.* (Ewing, Kariotis, and El-Mustafa, 1989) used an analytical finite element model utilizing a two-dimensional finite formulation with nonlinear stress-strain relations for the reinforced masonry. The masonry and reinforcement were modeled separately using an overlay model where the masonry was represented by a material with bi-modular orthotropy and the reinforcement was represented by an orthotropic material with a bilinear relationship. The overlay model was considered to be a rotating smeared crack model. They concluded that the FEM was a useful tool for the analysis of reinforced masonry building components subjected to monotonic loading.

2.3 Available Experimental Work

In studying the influence of aspect ratio (wall height/length ratio) on shear resistance of concrete block masonry walls, Woodward and Rankin (Woodward and Rankin, 1985) presented data from tests on seven ungrouted unreinforced masonry walls having different aspect ratios and subjected to in-plane lateral displacements in combination with various compressive vertical stress levels. They concluded that,

within the range of aspect ratios tested, the aspect ratio did not affect the lateral displacement at which diagonal cracking occurred when the wall behaviour was not influenced significantly by flexure. The behaviour of the walls after cracking, however, was affected by both the aspect ratio and the axial compressive stress.

Shing *et al.* (Shing, Noland, Klammerus, and Spaeh, 1989) examined the flexural and shear behaviour of reinforced masonry shear walls subjected to both monotonic and cyclic loads. The study was based on experimental results obtained from twenty 6 × 6 ft reinforced masonry panels tested under in-plane vertical and lateral loads. The deformation capability and the rate of load degradation of reinforced masonry shear walls under fully reversed displacement cycles were examined and a new formula for the nominal shear strength was proposed. This study indicated that the flexural strength and ductility of a reinforced masonry wall panel can be accurately evaluated by means of the simple flexure theory. Also, it was observed that the shear behaviour of a wall depends on several complicated mechanisms, such as the compression strut mechanism, the aggregate-interlock forces, and the resistance of the horizontal steel; additional research was recommended to develop a more realistic physical model for the shear mechanism.

As part of the US-TCCMAR program, an experimental study of T-section masonry walls under both pseudo static and dynamic test regimes was carried out by Priestley and Limin (Priestley and Limin, 1990); four flanged walls subjected to slow pseudo-static cyclic deformations were tested to failure. They investigated experimentally the influence of reinforcement content, flange width and mortar-bed confinement on the response of T-section walls.

The Japanese 5-storey full scale reinforced masonry building tested at the Building Research Institute in Tsukuba City, Japan, was a key component of the U.S.-Japan joint program on masonry buildings under seismic loads (Yamazaki *et al.*, 1988). The full scale test building was designed according to the new Japanese guidelines for medium rise reinforced masonry buildings. The test building was subjected to increasing cyclic load levels and a lateral load distribution according to the Japanese

Seismic Building Design Code. The crack pattern and yield development in the structure were described and the response of the building to a forced vibration test at discrete limit states was presented. The behaviour of the test building in the extended service load range was very stiff with only minor structural distress.

Antrobus *et al.* (Antrobus, Leiva, Merryman, and Klinger, 1988) have studied experimentally the in-plane seismic resistance of two-storey concrete masonry coupled walls with openings. The walls were loaded vertically by constant loads representing gravity loads, and horizontally by quasi-static reversed cyclic shear loads applied at the two floor levels. Their final report has not been completed at this time and very few data have been published.

Also, as a key step in the NSF (National Science Foundation) sponsored TCCMAR program, a 5-storey full-scale reinforced concrete masonry research building was tested under simulated seismic load (Seible *et al.*, 1993) to provide a benchmark for the validation and calibration of analysis and design models for masonry buildings in seismic zones. The building was tested from beginning to end utilizing simulated seismic load input rather than predetermined fixed lateral load patterns. The test demonstrated that masonry buildings in seismic zones can be designed with ductile performance characteristics in excess of the required design ductilities without any loss of lateral load carrying capacity.

2.4 Summary

The use of reinforced masonry as an engineered building material is of fairly recent origin. Therefore it is essential for designers to be thoroughly familiar with the properties and behaviour of masonry as well as the background of code provisions in order to be confident enough to take advantage of the full potential of masonry. As indicated in the literature review, very few attempts have been made in the field of modeling the behaviour of reinforced masonry shear walls and most these models are still in the developing stage. Also, some parameters affecting the performance of reinforced masonry shear walls are completely ignored in the experimental programs published to date. These shortcomings will be discussed and dealt with in the following chapters.

Chapter 3

ANALYTICAL MODELING

3.1 General

This chapter deals with the analytical side of the research where the capabilities and limitations of the finite element program **ADINA**, concerning the modeling of brittle materials such as concrete and masonry, will be discussed. Then, the new developments to introduce more features to the program will be outlined. Furthermore, calibration of the model will be carried out by analyzing plain and flanged reinforced masonry shear walls tested under lateral cyclic loading.

3.2 Finite Element Modeling

Program **ADINA** (**A**utomatic **D**ynamic **I**ncremental **N**onlinear **A**alysis) is a general purpose finite element program for linear and nonlinear, static and dynamic analysis which was originally developed at the Massachusetts Institute of Technology (Bathe, 1977). While the program has a lot of computing capabilities and has

been widely used in solving many practical problems, the program is limited when it deals with brittle material modeling such as for concrete or masonry.

The program has three basic features that are used to model the material behaviour for the concrete model: a nonlinear stress-strain relation to allow for the weakening of the material under increasing compressive stresses, failure envelopes that define cracking in tension as well as crushing in compression, and a strategy to model the post-cracking and crushing behaviour of the material.

The existing concrete model is a hypoelastic model that is based on a uniaxial stress-strain relation that is generalized to take biaxial and triaxial stress conditions into account. Concrete is considered as isotropic with an equivalent multiaxial Young's modulus when subjected to tension or low compression. If the material is under high compression, an orthotropic stress-strain matrix with the direction of orthotropy defined by the principal stress directions is employed. The concrete compressive stress-strain relationship implemented in the program is based on a model developed by Bathe and Ramaswamy (Bathe and Ramaswamy, 1979) and the tension failure is represented using a tension cut-off model.

Failure envelopes were employed to model the failure of the material in tension and compression in two and three-dimensional analyses and to account for multiaxial conditions in the uniaxial stress-strain behaviour. The shape of the compressive failure surface was based on experimental results reported by Kupfer *et al.* (Kupfer, Hilsdorf, and Rüschi, 1969) and Launay and Gachon (Launay, and Gachon, 1972). Also, it can be used to represent a large number of different envelopes like the biaxial envelope of Liu *et al.* (Liu, Bilson, and Slate, 1972) and the triaxial envelope of Khan and Saugy (Khan and Saugy, 1972). Failure envelopes were employed to establish the uniaxial stress-strain law accounting for multiaxial stress conditions, and to identify whether tensile or crushing failure of the material has occurred.

The concrete model in ADINA is a smeared fixed crack model where the crack orientation is initially established normal to the principal tensile stress and will

have this orientation during the whole incremental load/displacement-time history. Reinforcement can be modeled by discrete truss elements or isotropic plane stress elements and employing an elasto-plastic stress-strain relationship.

While the computer program ADINA contains many features that make it a very powerful numerical tool, it has some limitations concerning brittle material modeling. Some of these limitations are listed below:

- the available model is a fixed smeared crack model, therefore, it can not realistically represent the crack rotation under incrementally increased loading (or unloading) processes where the crack orientation is already fixed when the crack is originally initiated.
- the available concrete model is implemented to work only on a three-dimensional brick element and on a plane stress element. Using a three-dimensional brick element to achieve large scale computations, such as modeling a full shear wall structure with intersecting and flanged shear walls, will result in a very expensive solution even if it is numerically possible.
- the available concrete model does not account for the tension stiffening that develops in a cracked material due to aggregate interlock. Only a simple tension cut-off model is employed in the original program.
- reinforcement can only be modeled by discrete truss elements or by in-plane isotropic elements. Therefore, reinforcement with different ratios in orthogonal directions can only be modeled using general truss elements.

In summary, the program ADINA is lacking almost all the new developments that are associated with brittle material modeling despite its powerful numerical capabilities in solving dynamic and static related problems. Because of the aforementioned limitations, more developments and enhancements need to be implemented in the finite element program ADINA. Hence as part of this thesis a new model to predict the nonlinear behaviour of reinforced masonry up to maximum load and able to capture the ultimate degradation as well as softening behaviour of masonry is developed. The following are some of the developments as part of

this thesis:

- Vecchio and Collins's Modified Compression Field Theory (MCFT) is implemented where a rotating smeared crack concept is employed.
- Coaxial and non-coaxial rotating smeared crack concepts are introduced.
- Various tension-stiffening models are implemented for both fixed and rotating crack models to account for the post-cracking of reinforced masonry.
- Various constitutive models for masonry under compression are employed that are based on available experimental work on masonry and available concrete models in the literature.
- Force transmitting mechanisms between masonry and reinforcing bars at a crack surface are modeled. Also, the shear transfer mechanism by aggregate interlock is modeled.
- Program ADINA is modified to ensure that all the available masonry models are working on shell elements. This permits the modeling of complex three-dimensional problems.
- Reinforcement is modeled by a plane stress element where an orthotropic material constitutive law is assumed and a zero value is assigned for the shear modulus as well as Poisson's ratio.
- Cyclic loading capabilities are also added where the failure status including opening and closing of a crack and crushing of the material at each Gauss point is updated for every load/displacement increment.

In the following subsections, different constitutive relations for uncracked/cracked masonry and reinforcement along with different strategies associated with the post-cracking and post-crushing behaviour modeling will be discussed.

3.2.1 Masonry Compressive Stress-Strain Relationships

a. Monotonic Loading

A detailed knowledge of the stress-strain behaviour of concrete masonry is necessary to predict the performance of masonry structures responding inelastically

under seismic loads. In pursuing a nonlinear analysis for a reinforced masonry structure, the accuracy of the physical behaviour representation depends mainly on the material model used to simulate the reinforced masonry behaviour. For concrete under compressive action, the stress-strain relationship has been investigated experimentally by many researchers and many analytical models have been developed. Based on these models, different problems have been approached and good agreement between the analytical and experimental results has been found.

While extensive experimental data for concrete under uniaxial, biaxial, and triaxial compressive action have been reported, little experimental data have been published for masonry. Based on the available data for concrete masonry, it was concluded that these data follow, to a great extent, the same models previously developed for concrete. Therefore, a few researchers recently have modeled some masonry assemblages utilizing the adapted concrete models and good agreement was achieved between the experimental and numerical results.

In the current study, different material constitutive models, developed mainly for concrete, were adapted for masonry and incorporated in parallel with experimentally based ones into the numerical analysis study on masonry assemblages tested experimentally. Model sensitivity analyses were carried out to study the influence of using different models on the overall physical behaviour of masonry structural elements under complex states of stress. The following three constitutive models are of special interest:

- Bathe and Ramaswamy (Bathe and Ramaswamy, 1979) have derived a stress-strain relation for concrete or rock material based on tests for concrete under uniaxial compressive stress (Kupfer *et al.*, 1969, and Saenz, 1964). This model was adapted for masonry where the principal compressive stress-strain law of grouted masonry (f_m, ϵ_m) was assumed as

$$f_m = \frac{f'_m(E_o/E_s)(\epsilon_m/\epsilon_o)}{1 + A(\epsilon_m/\epsilon_o) + B(\epsilon_m/\epsilon_o)^2 + C(\epsilon_m/\epsilon_o)^3} \quad (3.1)$$

where

$$A = \frac{[E_o/E_u + (p^3 - 2p^2)E_o/E_s - (2p^3 - 3p^2 + 1)]}{[(p^2 - 2p + 1)p]}, \quad B = [(2E_o/E_s - 3) - 2A],$$

and $C = [(2 - E_o/E_s) + A]$

The strength parameters $E_o, f'_m, \epsilon_o, E_s = f'_m/\epsilon_o, f_{um}, \epsilon_{um}, p = \epsilon_{um}/\epsilon_o$ and, $E_u = f_{um}/\epsilon_{um}$ are obtained from uniaxial tests where

ϵ_o = strain at uniaxial peak compressive stress

f_{um}, ϵ_{um} = uniaxial ultimate compressive stress and corresponding strain.

Employing failure envelopes to establish the uniaxial stress-strain law accounting for multiaxial stress conditions, the crushing stress value is calculated as $\tilde{f}'_m$, and a ratio $\gamma_1 = \tilde{f}'_m/f'_m$ can be determined. Therefore, one can find:

$$\tilde{f}_{um} = \gamma_1 f_{um}, \tilde{\epsilon}_o = \gamma_1 \gamma \epsilon_o, \text{ and } \tilde{\epsilon}_{um} = \gamma_1 \gamma \epsilon_{um}$$

where γ is a constant and $\tilde{f}'_m, \tilde{f}_{um}, \tilde{\epsilon}_o$, and $\tilde{\epsilon}_{um}$ are employed in order to establish the uniaxial stress-strain law under multiaxial conditions. The increase of strength parameters for the model under multiaxial conditions is defined in Fig. 3.1.

• A series of tests by Vecchio and Collins (Vecchio and Collins, 1986) demonstrated that the concrete compressive strength is greatly reduced when coexisting principal tensile strains are high. This resulted in the development of the relationship between the principal compressive stress and principal compressive strain for concrete as shown in Fig. 3.2. This model was adapted for masonry where the principal compressive stress-strain law of grouted masonry (f_m, ϵ_m) for the ascending branch was assumed as

$$f_m = f'_m / \beta [2(\frac{\epsilon_m}{\epsilon_o}) - (\frac{\epsilon_m}{\epsilon_o})^2] \quad (3.2)$$

where f'_m and ϵ_o are the uniaxial peak compressive stress and corresponding strain of grouted masonry and

$$\beta = 0.8 - 0.34(\frac{\epsilon_{m1}}{\epsilon_o}) \geq 1 \quad (3.3)$$

where ϵ_{m1} is the tensile principal strain in the direction normal to that being con-

sidered. In Eq. 3.3, the empirical constants were developed originally for concrete under biaxial states of stress. Since there are no experimental data available for concrete masonry under biaxial states of stress and since the masonry is fully grouted, it is reasonable to employ the same equation (Eq. 3.3) for concrete masonry.

For the descending branch, the compressive stress-strain constitutive relationship for concrete developed by Vecchio and Collins (Vecchio and Collins, 1986) was adapted for masonry as

$$f_m = f'_m / \beta [0.175x^3 - 0.525x^2 + 1] \quad (3.4)$$

in which

$$x = \frac{2(\epsilon_{mn} - 1)}{(\epsilon_{mnu} - 1)}$$

where ϵ_{mn} is the normalized strain at maximum compressive stress, $\epsilon_{mn} = \epsilon_m / \epsilon_o$, and ϵ_{mnu} is the normalized ultimate compressive strain, $\epsilon_{mnu} = \epsilon_{mu} / \epsilon_o$. The parameter ϵ_{mu} is the strain at $0.1 f'_m$ at which the cubic relation levels off. Vecchio and Collins anticipated that the inclusion of the effects of confinement will lead to better results. Therefore, the same parabola (Eq. 3.2) and cubic relations (Eq. 3.4) are employed with the masonry reaching a maximum compressive stress of $f_m = k_1 f'_m$ at a strain of $k_2 \epsilon_o$. The parameters k_1 and k_2 were estimated from the confinement models proposed by Sheikh and Uzumeri (Sheikh and Uzumeri, 1982) but were modified to suit the current analysis.

• Hart *et al.* (Hart, Sajjad, Kingsley, and Noland, 1989) have developed a "Best Fit" mathematical model to represent the idealized stress-strain behaviour of both confined and unconfined grouted masonry in compression. The model, based on experimental data from recent prism tests, account for various types of confinement reinforcing. The ascending part of the curve can be written in terms of one shape parameter, C , and two material parameters, f'_m and ϵ_o as:

$$f_m = f'_m [C \epsilon_m^2 + (\frac{1}{\epsilon_o} - C \epsilon_o) \epsilon_m] \quad (3.5)$$

and the equation of the descending part of the curve can be written in terms of two shape parameters, D and E , and two material parameters f'_m and ϵ_{um} as:

$$f_m = f'_m [D + (1 - D)e^{E(\epsilon_o - \epsilon_m)}] \quad (3.6)$$

Thus there are three shape parameters (C , D , and E) and two material parameters (f'_m and ϵ_o) that must be defined in order to completely describe the stress-strain curve using the Best Fit model.

Different confinement reinforcements have been tested (see Ref. Hart, Sajjad, Kingsley, and Noland, 1989, for more details) and their effect on the inelastic behaviour of concrete masonry has been monitored. Among these different types is the one based on the new criteria for the strength design of masonry shear walls proposed for the 1988 Uniform Building Code (UBC) where the minimum confinement reinforcement is recommended in Section 2412. Therefore, Fig. 3.3 illustrates stress-strain relationships related only to two types of prisms used for the aforementioned study (Hart, Sajjad, Kingsley, and Noland, 1989): the unreinforced concrete masonry (URM) prisms and prisms reinforced according to the Uniform Building Code proposal. Table 3.1 summarizes the various shape parameters involved in the Best Fit model.

3.2.2 Cyclic Constitutive Compressive Characteristics

The basic assumptions of the orthogonally anisotropic models presented in Section 3.2.1 were retained for cyclic behaviour with the inclusion of path dependency in the stress-strain relationships for masonry and steel. Due to the lack of experimental data on the behaviour of grouted masonry under cyclic loading, the cyclic constitutive model for concrete under uniaxial cyclic loading by Reinhardt et al. (Reinhardt and Cornelissen, 1984) was adapted for concrete masonry and incorporated into the numerical analysis study on masonry assemblages tested experimentally.

As mentioned earlier, the principal directions were considered independently

in the tension-tension and compression-compression domains while in the tension-compression domain the coupling effect was represented in the damage factor β that was modified as

$$\beta = 0.8 - 0.34\left(\frac{\epsilon_{m1}^c}{\epsilon_o}\right) \geq 1 \quad (3.7)$$

where ϵ_{m1}^c is the absolute tensile principal strain in the direction normal to that being considered and can be expressed as $\epsilon_{m1}^c = \epsilon_{m1} - \epsilon_{mr}$ where ϵ_{mr} is the residual strain under unloading in compression as shown in Fig. 3.4. Unloading and reloading under compression follows constant stiffness formulations up to the envelope. Before the peak, the elastic modulus of grouted concrete masonry, E_m , is assumed for unloading and reloading. In the post-peak region, the stiffness is calculated based on the residual strain ϵ_{mr} that can be calculated from the following relationship by Reinhardt et al.

$$\epsilon_{mr} = \epsilon_m \frac{[1 - 0.425\exp(1 - \frac{\epsilon_m}{\epsilon_o})]}{[1 + \frac{\epsilon_m}{\epsilon_o}\exp(1 - \frac{\epsilon_m}{\epsilon_o})]} \quad (3.8)$$

In Eq. 3.8, the empirical constants were developed originally for concrete under uniaxial states of stress. Since there are no experimental data available for concrete masonry under biaxial states of stress and since the masonry is fully grouted, it is reasonable to employ Eq. 3.8 for concrete masonry.

3.2.3 Tension-Stiffening Models for Masonry

Similar to concrete, as the masonry reaches its tensile strength, primary cracks form and their number and extent are controlled by the size and placement of the reinforcing steel. At the primary cracks, the masonry stress drops to zero and the steel carries the full load. However, the masonry between cracks still maintains some tensile resistance which will drop as the load increases. The drop will be associated with the breakdown in bond and a secondary system of internal cracks will develop around the reinforcement.

The tension stiffening model effect has been represented using two procedures. In the first one, the tension portion of the masonry stress-strain curve has been given a descending branch. In the second procedure, the tension stiffening effect is represented by increasing the steel stiffness in a modified stress-strain relationship for steel. The additional stress in the steel represents the total tensile force carried by both the steel and concrete masonry between the cracks. The former procedure is the most commonly used by different researchers and, therefore, will be used in the current study; note that the released forces at cracked masonry will be transmitted to the steel reinforcement and then the latter will be checked against yield.

While little experimental data for concrete under uniaxial tensile action have been reported, no experimental data are available for concrete masonry under similar action. In the current study, different material constitutive models, developed mainly for concrete, are adapted for concrete masonry and incorporated into the numerical analysis study on masonry assemblages tested experimentally. Model sensitivity analysis will be carried out to study the influence of using different models on the overall physical behaviour of masonry structural elements under complex states of stress.

Few accurate models are available to describe macroscopically the softening response of concrete under uniaxial tension. Scanlon (Scanlon, 1971) and Lin and Scordelis (Lin and Scordelis, 1975) have used approximate formulations to incorporate gradual loss of stress-carrying capacity of plain concrete in numerical schemes that account for the tension stiffening effect observed in reinforced concrete. As illustrated in Fig. 3.5a, a tension stiffening effect was first introduced by Scanlon as a step reduction in the tensile stress. Lin and Scordelis have assumed a cubic polynomial relationship between post-cracking peak stress and strain as illustrated in Fig. 3.5b. In fracture-based studies, Bazant and Oh (Bazant and Oh, 1983) have proposed a linear-softening behaviour with a negative softening modulus which is related to the fracture energy, peak stress, initial modulus and a characteristic

width of nonlinear zone which can be seen in Fig. 3.5c. Gylltoft (Gylltoft, 1983) has assumed a bilinear stress-strain relationship in the post-peak region as can be seen in Fig. 3.5d. This relationship is dependent upon the fracture energy, initial modulus, peak stress, and a parameter called characteristic length. Peterson (Peterson, 1981) has proposed a similar bilinear stress-crack width relationship rather than the conventional stress-strain relationship. Also, Mazars (Mazars, 1981) has proposed a continuous damage-based formulation to represent post-peak stress-strain curves of concrete. An internal state damage variable was considered as zero prior to peak stress and a function of strain thereafter. The two aforementioned relationships are illustrated in Figs. 3.5e and 3.5f, respectively.

After reviewing different tension stiffening models and experimental evidence on the post-cracking behaviour of brittle materials, three models were considered in the numerical implementation. First, a biaxial tensile response model developed by Stevens *et al.* (Stevens, Uzumeri, and Collins, 1987) for a biaxial case with varying principal directions and different amounts of orthotropic reinforcing will be introduced in detail. Then, a commonly used tension stiffening model developed by Vecchio and Collins (Vecchio and Collins, 1988) will be briefly mentioned. Finally, a third model is proposed that is based on experimental evidence by Gopalaratnam and Shah (Gopalaratnam and Shah, 1985) and on the crack band theory by Bazant and Oh (Bazant and Oh, 1983).

• Stevens *et al.* (Stevens, Uzumeri, and Collins, 1987) have developed a post-cracking model for reinforced concrete where the tensile stress-strain descending branch after peak tensile stress was represented by the following equation:

$$f_m = f_{mcr}[\alpha + (1 - \alpha)e^{-\lambda_t(\epsilon_m - \epsilon_{mcr})}] \quad (3.9)$$

where for n groups of reinforcing with properties ρ_{si} , d_{bi} , and oriented at θ_i to the x axis:

$$\alpha = \sum_{i=1}^n [\cos^2 \Delta \theta_i + \sin^2(\frac{4\Delta \theta_i^2}{\pi})] \alpha_i$$

$$\lambda_t = \frac{270}{\sqrt{\alpha}} \text{ and } \lambda \leq 1000$$

$$\alpha_i = C_t \frac{\rho_s}{d_{bi}}$$

ρ_s and d_b are the reinforcement percentage and the diameter of reinforcement bars, respectively

$$C_t = \text{constant} = 75 \text{ mm (empirical constant that has a unit of length)}$$

$\Delta\theta_i = \theta - \theta_i$ in which θ_i is the orientation of the reinforcing group and θ is the principal tensile strain direction

f_m and ϵ_m are the stress and strain in the principal direction oriented at θ to the x axis.

As can be seen in this model, the orthotropy of the steel reinforcement, the bond between the concrete and steel and their influence on the tension stiffening are represented in the model as encountered in the factor α . The tensile stress-strain relationship of Eq. 3.9 is illustrated in Fig. 3.6.

• Then, Vecchio and Collins (Vecchio and Collins, 1988) presented a simpler model for concrete where the stress-strain relationship for the descending branch after cracking is suggested in the following form:

$$f_m = \frac{f_{mcr}}{1 + \sqrt{200\epsilon_{m1}}} \quad (3.10)$$

where f_m and ϵ_{m1} are the stress and strain in the principal direction.

• Gopalaratnam and Shah (Gopalaratnam and Shah, 1985) have proposed an analytical model which assumes a unique stress-strain relationship only in the ascending part and a unique relationship only between stress and crack width in the descending part. This was deduced from results obtained from an experimental program that was carried out on different concrete, mortar, and paste specimens subjected to tensile stress.

For the ascending part, a simple expression that has been used for defining compressive behaviour of concrete is recommended (Shah *et al.*, 1983) for the uniaxial tensile stress-strain curve up to the peak stress value:

$$f_m = f_{mcr} \left[1 - \left(1 - \frac{\epsilon_m}{\epsilon_{mcr}} \right)^A \right] \quad (3.11)$$

where

f_m = tensile stress

f_{mcr} = peak value of f_m

ϵ_m = tensile strain

ϵ_{mcr} = value of strain, ϵ_m , at f_{mcr}

$A = E_t \epsilon_{mcr} / f_{mcr}$

E_t = initial tangent modulus.

For the descending part, due to the localized nature of the post-peak deformations, no unique stress-strain relationship existed. Instead, Gopalaratnam and Shah found a unique relationship between average stress and average crack width in the form:

$$f_m = f_{mcr} (e^{-kw^\lambda}) \quad (3.12)$$

where w is the crack width, and k and λ are constants.

Due to the localized nature of the post-peak deformation, Gopalaratnam and Shah could not find a unique average tensile stress-strain relationship for concrete, mortar, and paste specimens where the strains were calculated based on different strain gage lengths. Bazant and Oh (Bazant and Oh, 1983) have proposed the *crack band model* where the front of an advancing crack band (microcrack zone), called the fracture process zone, has a certain characteristic width w_c which is a function of the maximum aggregate size. Now a new term, *fracture process zone strain*, ϵ_f , can be defined as the ratio between the sum of individual microcracks openings, δ_f , and the crack band width w_c .

$$\epsilon_f = \frac{\delta_f}{w_c} \quad (3.13)$$

where

$$w_c = \eta d_a \quad (3.14)$$

where

w_c = crack band width

η = constant (between 1.5 and 4)

d_a = maximum aggregate size.

Therefore, based on results by Gopalaratnam and Shah, and on the existing relationship between the post-peak cracking stress and crack width, and utilizing the *crack band model* of the fracture process zone developed by Bazant and Oh, the total strain of a domain loaded in tension is equal to the sum of three main components as follows:

$$\epsilon_t = \epsilon_e + \epsilon_{mp} + \epsilon_f \quad (3.15)$$

where

$$\epsilon_e = \frac{f_m}{E_t}$$

$$\epsilon_{mp} = \epsilon_{mcr} - \frac{f_{mcr}}{E_t}$$

$$\epsilon_f = \frac{1}{w_c} \left(-\frac{1}{k} \ln \frac{f_m}{f_{mcr}} \right)^{\frac{1}{\lambda}}$$

and

$$\epsilon_t = \frac{f_m}{E_t} + \epsilon_{mcr} - \frac{f_{mcr}}{E_t} + \frac{1}{w_c} \left(-\frac{1}{k} \ln \frac{f_m}{f_{mcr}} \right)^{\frac{1}{\lambda}} \quad (3.16)$$

where different terms are as defined before and different strain components are illustrated in Fig. 3.7.

It should be mentioned here that, for concrete masonry, the cracking stress depends on the orientation of the principal tensile stress relative to the mortar bed joints. The tensile failure stress is related to the angle ϕ , measured from the vertical axis, by

$$f(\phi)_{mcr} = f_{mcr}(1.0 + 0.006\phi - 7.8 \times 10^{-5}\phi^2) \quad (3.17)$$

where f_{mcr} is the cracking stress for uniaxial tension normal to the bedjoint, and ϕ is in degrees. The relation is normalized from an empirical equation suggested by Nunn (Nunn, 1980).

3.2.4 Cyclic Constitutive Tensile Characteristics

Gopalaratnam and Shah (Gopalaratnam and Shah, 1985) have extended their experimental work on the softening response of plain concrete and mortar to include specimens cyclically loaded under tension. Results of concrete and mortar specimens showed that a linear relationship exists between the displacement at which unloading commenced δ_{un} and the residual displacement δ_r . The slope of this linear relationship is close to one. Partly based on the experimental observations, they have proposed the following relationship between δ_{un} and δ_r

$$\delta_{un} = a + \delta_r \quad \text{for } \delta \geq \delta_{mcr} \quad (3.18)$$

where they assumed that the unloading slope is equal to the initial tangent stiffness K_t , then $a = \frac{f_m}{K_t}$. By adapting the same relationship for concrete masonry and by comparing Eqs. 3.16 and 3.18, the residual strain can be expressed as follows

$$\epsilon_r = \epsilon_{mcr} - \frac{f_{mcr}}{E_t} + \frac{1}{w_c} \left(-\frac{1}{k} \ln \frac{f_m}{f_{mcr}} \right)^{\frac{1}{n}} \quad (3.19)$$

Although no cyclic experiments were conducted in the prepeak region, they proposed the following relationship between δ_{un} and δ_r for the prepeak region as follows

$$\delta_{un} = b\delta_r \quad \text{for } \delta \leq \delta_{mcr} \quad (3.20)$$

where

$$b = \frac{K_t}{K_t - \frac{f_m}{K_t}} \quad (3.21)$$

By adapting the same relationship for concrete masonry, similar relationship between the unloading strain ϵ_{un} and residual strain ϵ_r for the prepeak region can be expressed as follows

$$\epsilon_{un} = \epsilon_r \left[\frac{\epsilon_{mcr}}{\epsilon_{mcr} - \frac{f_m}{E_m}} \right] \quad (3.22)$$

Various strain parameters are illustrated in Fig. 3.8.

3.2.5 Cyclic Constitutive Law for Reinforcing Steel

The monotonic and cyclic stress-strain relationships of reinforcing steel are illustrated in Fig. 3.9. Reinforcing steel can be modeled explicitly by two- or three-dimensional truss elements or can be represented as a smeared layer of steel having an equivalent thickness to represent the same area of steel per unit length as provided by the discrete reinforcing bars. In this study, the second option was chosen and the steel was modeled as a smeared layer and no coupling effect between the orthogonal layers was assumed. Also, the hardening effect is modeled by assuming a residual factor, α , representing a percentage of the steel's initial stiffness.

3.2.6 Transmitting Loads Across Cracks

Dealing with average stress in concrete or reinforcement results in ignoring local concentrations of stresses and, accordingly, force transmitting mechanisms between concrete and reinforcement. At a crack, the tensile stresses in reinforcement are expected to be higher than average while midway between cracks the tensile stresses will be lower than average. On the other hand, the concrete tensile stresses will be zero at a crack and higher than average midway between cracks. These local variations may govern the reinforcement's ability to transmit tension across the cracks and accordingly the ultimate capacity of a biaxially stressed element.

A comparison between the average stresses (Plane 1) and the actual local stresses that occur at a crack (Plane 2) is illustrated in Fig. 3.10. The critical crack direction is assumed normal to the principal tensile strain direction which coincides with that of the principal tensile stress. The calculated average shear stress on Plane 1 is zero while local shear stresses v_{mi} exist and may be accompanied by small local compressive stresses, f_{mi} , across the crack. As illustrated in Figs. 3.10b and 3.10c, the two sets of stresses must be statically equivalent since the applied stresses f_x , f_y , and v_{xy} are fixed. Therefore, assuming a unit area for both Plane 1 and Plane 2, both sets of internal stresses should produce the same force in the x-direction. Hence,

$$\rho_{sx} f_{sx} \sin\theta + f_{m1} \sin\theta = \rho_{sx} f_{sxcr} \sin\theta - f_{m1} \sin\theta - v_{m1} \cos\theta \quad (3.23)$$

and for the same reason in the y-direction

$$\rho_{sy} f_{sy} \cos\theta + f_{m1} \cos\theta = \rho_{sy} f_{sy cr} \cos\theta - f_{m1} \cos\theta + v_{m1} \sin\theta \quad (3.24)$$

Rearranging Eq. 3.23 as follows:

$$\rho_{sx} (f_{sxcr} - f_{sx}) = f_{m1} + f_{m1} - v_{m1} / \tan\theta \quad (3.25)$$

and Eq. 3.24 as follows:

$$\rho_{sy} (f_{sy cr} - f_{sy}) = f_{m1} + f_{m1} - v_{m1} \tan\theta \quad (3.26)$$

From Eqs. 3.25 and 3.26, the following equation can be derived:

$$f_{m1} = \frac{1}{2} \rho_{sy} (f_{sy cr} - f_{sy}) + \frac{1}{2} \rho_{sx} (f_{sx cr} - f_{sx}) - f_{m1} + (1 - \frac{1}{\tan\theta}) \frac{v_{m1}}{2} \quad (3.27)$$

However, the stress in the reinforcement at a crack cannot exceed the yield strength f_{yx} , and f_{yy} , therefore, the average tensile stress should satisfy the following condition with no shear stress and no compressive stresses on the crack:

$$f_{m1} \leq \rho_{sy}(f_{yy} - f_{sy})\sin^2\theta + \rho_{sz}(f_{yz} - f_{sz})\cos^2\theta \quad (3.28)$$

As the cracking usually occurs along the interface between the cement paste and the aggregate particles, the resulting rough cracks can transfer shear by aggregate interlock which in turn is a function of aggregate size as illustrated in Fig. 3.11. Walraven (Walraven, 1981) has studied the relationship between the shear across the crack v_{mi} , the crack width w , and the compressive stress on the crack; the following relationships were derived:

$$v_{mi} = 0.18v_{mimax} + 1.64f_{mi} - 0.82\frac{f_{mi}^2}{v_{mimax}} \quad (3.29)$$

where

$$v_{mimax} = \frac{\sqrt{-f'_m}}{0.31 + 24w/(a + 16)} \quad (3.30)$$

where a is the maximum aggregate size in millimeters and the stresses are in MPa. The numerator of Eq. (3.30) should be multiplied by 12, and 16 in the denominator should be replaced by 0.63 if inch and psi units are being used.

The crack width w should be the average crack width over the crack surface. It could be taken as the product of principal tensile strain and the crack spacing S_θ as follows:

$$w = \epsilon_1.S_\theta \quad (3.31)$$

where

$$S_\theta = \frac{1}{\frac{\sin\theta}{S_{mx}} + \frac{\cos\theta}{S_{my}}} \quad (3.32)$$

where S_{mx} and S_{my} are the indicators of the crack control characteristics of the x-reinforcement and the y-reinforcement, respectively.

Again, the calculated average tension f_{m1} is subjected to the condition:

$$f_{m1} \leq v_{mimex}(0.18 + 0.3k^2)\tan\theta + \rho_{sy}(f_{yy} - f_{sy}) \quad (3.33)$$

where $k = (1.64 - 1/\tan\theta)$.

The solution technique for determining the response of a biaxially stressed element can be seen in details elsewhere (Vecchio and Collins, 1986).

3.2.7 Constitutive Relations

The biaxial constitutive equations for reinforced grouted masonry are based on Bathe's orthogonally anisotropic model (Bathe and Ramaswamy, 1979) and on Vecchio and Collins's Modified Compression Field Theory (Vecchio and Collins, 1986). This model can be classified as a smeared crack model where fixed and rotating crack approaches are employed. Cracks are assumed to be well distributed across the elements. Both materials, masonry and steel, are assumed to be perfectly bonded and subjected to the same strain field. The incremental stress-strain relation in the local n-s coordinates can be expressed as follows:

$$d\sigma = D'd\epsilon; D' = D'_m + D'_s \quad (3.34)$$

where D'_m is the tangent constitutive matrix of grouted masonry, D'_s is the tangent constitutive matrix of smeared reinforcement, $\sigma = \{\sigma_n \ \sigma_s \ \sigma_{ns}\}^T$ and $\epsilon = \{\epsilon_n \ \epsilon_s \ \epsilon_{ns}\}^T$.

Uncracked Masonry

Let σ_{p1} and σ_{p2} be the principal stresses with $\sigma_{1,2} \leq \sigma_{p1}$ and E_{p1} and E_{p2} the corresponding uniaxial Young's moduli. The uncracked masonry is considered as isotropic with an equivalent multiaxial Young's modulus when subjected to tension or low compression, where such a state is defined by $\sigma_{p2} \geq \kappa\sigma'$. The variable κ is typically 0.4 and the equivalent multiaxial Young's modulus, E , is then obtained using the following weighting scheme:

$$E = \frac{\sigma_{p1}E_{p1} + \sigma_{p2}E_{p2}}{\sigma_{p1} + \sigma_{p2}} \quad (3.35)$$

and Poisson's ratio is assumed to be constant.

Then, the tangent stiffness matrix D for uncracked masonry is assumed to have the following form:

$$D'_m = \frac{E}{(1 - \nu^2)} \begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix} \quad (3.36)$$

If the masonry is under high compression, i.e., $\sigma_{p2} \leq \kappa\sigma'$, an orthotropic stress-strain matrix with the direction of orthotropy defined by the principal stress directions is employed. The stress-strain matrix corresponding to these directions is

$$D'_m = \frac{1}{(1 - \nu^2)} \begin{bmatrix} E_1 & \nu E_{12} & 0 \\ \nu E_{12} & E_2 & 0 \\ 0 & 0 & \frac{1-\nu}{2} E_{12} \end{bmatrix} \quad (3.37)$$

where ν is the constant Poisson's ratio, and the shear modulus in a coordinate plane is calculated from the weighted Young's modulus corresponding to that plane as

$$G_{12} = \frac{E_{12}}{2(1 + \nu)} = \frac{1}{2(1 + \nu)} \frac{\sigma_{p1}E_{p1} + \sigma_{p2}E_{p2}}{\sigma_{p1} + \sigma_{p2}} \quad (3.38)$$

Cracked Masonry

Tensile failure occurs if the tensile stress in a principal stress direction exceeds the tensile failure strength and a plane of failure develops perpendicular to the principal stress direction. This material failure will result in reductions in the normal and shear stiffness across the plane of failure and, then, the corresponding normal stress is released. Assuming that the principal stress σ_{p1} is larger than the tensile failure stress, the new material stress-strain relation is

$$D'_m = \frac{E}{(1-v^2)} \begin{bmatrix} \eta_n & v\eta_n & 0 \\ v\eta_n & 1 & 0 \\ 0 & 0 & \eta_s \frac{1-v}{2} \end{bmatrix} \quad (3.39)$$

Now, assuming that the principal stress σ_{p2} is larger than the tensile failure stress, the new material stress-strain relation that represents two perpendicular cracks is

$$D'_m = \frac{E}{(1-v^2)} \begin{bmatrix} \eta_n & v\eta_n & 0 \\ v\eta_n & \eta_n & 0 \\ 0 & 0 & \eta_s \frac{1-v}{2} \end{bmatrix} \quad (3.40)$$

where, typically, $\eta_n = 0.001$ and $\eta_s = 0.5$. The factor η_n is not set equal to zero in order to avoid the possibility of a singular stiffness matrix while the value employed for η_s depends on a number of physical factors.

In Eqs. 3.39 and 3.40, the shear modulus for cracked masonry G_{ns} was defined as

$$G_{ns} = \eta_s G \quad (3.41)$$

in which G is the elastic shear modulus and η_s is a shear retention factor that varies from one to zero and is assigned a value of 0.5 for those equations.

Rots and Blaauwendraad (Rots and Blaauwendraad, 1989) assumed for η_s a linear and nonlinear function of the tensile strain ϵ_1 as follows:

$$\eta_s = \left(1 - \frac{\epsilon_1 - \epsilon_{cr}}{\epsilon_u - \epsilon_{cr}}\right)^p \quad (3.42)$$

in which ϵ_1 is the crack normal strain at the beginning of the load increment, ϵ_{cr} is the strain at which a crack initiates and ϵ_u is the tensile strain at which the shear retention factor is assumed to be zero. Fig. 3.12 illustrates the function η_s of Eq. 3.42 for p equal to 1 and 2.

The aforementioned representation of the shear modulus G_{ns} will result in the loss of coaxiality between the principal stress and strain directions. To maintain

the coaxiality in a coaxial rotating crack model, G_{ns} has to be expressed in terms of the principal stresses and strains as follows:

$$G_{ns} = \frac{\sigma_{p1} - \sigma_{p2}}{2(\epsilon_{p1} - \epsilon_{p2})} \quad (3.43)$$

Reinforcement

The constitutive matrix for the reinforcement is given by:

$$D'_s = \begin{bmatrix} \rho_n E_{sn} & & \\ & \rho_s E_{ss} & \\ & & 0 \end{bmatrix} \quad (3.44)$$

in which D'_s is the tangent constitutive matrix of smeared reinforcement, ρ_n and ρ_s are the reinforcement ratios in the n, s directions, and E_n, E_s are the tangent moduli of steel in the n, s directions obtained from a bilinear stress-strain relationship.

Coordinates Transformation

The tangent stiffness for masonry and reinforcement in the global coordinates can be obtained by coordinate transformation as follows:

$$D_m = T^T \cdot D'_m \cdot T \quad (3.45)$$

and

$$D_s = T^T \cdot D'_s \cdot T \quad (3.46)$$

in which T is the transformation matrix that is the function of the angle between the global coordinate axes and the axes of orthotropy β and

$$T = \begin{bmatrix} c^2 & s^2 & cs \\ s^2 & c^2 & -cs \\ -2cs & 2cs & c^2 - s^2 \end{bmatrix} \quad (3.47)$$

where $c = \cos\beta$ and $s = \sin\beta$.

3.2.8 Orthotropy of Concrete Block Masonry

Unlike concrete, masonry is highly orthotropic. As illustrated in Fig. 3.13, masonry can be considered as a sandwich consisting of three layers with the two outer skins representing the block faceshells plus mortar head and bed joints, and the core representing the grout and the vertical webs of the blocks.

Since the mortar and face shells in the outer skins have a similar basic composition and close mechanical properties and also since mortar joints constitute a relatively small part of the skin, it is reasonable to assume that the outer skins are isotropic and have the same properties as those of the ungrouted masonry. On the other hand as illustrated in Fig. 3.13, the inner core is highly orthotropic due to the cross webs of the blocks extending only in the vertical direction and the degree of orthotropy also depending on the percentage of grouting.

For the inner core, the grout columns and cross webs of blocks are the two main contributors to the stiffness in the vertical direction. The initial tangent stiffness of the orthotropic inner layer in the vertical direction can be expressed by the following formula:

$$E_v = a_{bm}E_{bm} + a_gE_g \quad (48)$$

where E_{bm} and E_g are initial elastic modulus of block material and grout, respectively, and a_{bm} and a_g are factors representing the fraction of sectional area of block cross webs and grout, respectively; the latter are functions of the block dimensions and number of grouted cores per unit length of the wall. They can be expressed by the following formulas

$$a_{bm} = c \cdot n_c \cdot t_w \quad (49)$$

$$a_g = t_g/L_g \quad (50)$$

where

t_w = thickness of the cross webs of blocks

c = reduction factor

n_c = number of cross webs per unit length of the wall

t_g = thickness of grouted cores

L_g = spacing of grouted cores.

The reduction factor c is to account for the discontinuity of some of the webs. A value of 0.5 appears reasonable for typical ungrouted and partially grouted walls assuming that 50 % of the webs are discontinuous and a value close to one is reasonable for fully grouted walls where good monolithic action can be achieved. The contribution of the discontinued cross webs to the vertical stiffness due to their monolithic attachment to the face shells is ignored in the previous assumption since there is no significant effect of this factor on the overall vertical stiffness of the full wall section.

In the horizontal direction, two cases were considered. In the first case where all the cores are fully grouted, the inner core is considered to be isotropic with an initial elastic modulus equal to the smaller of that of grout and block material. In the second case where the masonry is partially grouted, there is a contribution from the grout to the horizontal stiffness through shear transfer since the grout columns are bonded to the face shells and also due to their bridging effect on the surrounding cross webs. Test results indicate that the stresses change in a very minor way and can be ignored; therefore, almost no contribution from the inner core to the horizontal stiffness is assumed in the second case and a value of 2 % of the vertical stiffness is assumed for the horizontal stiffness.

Unlike isotropic materials, the shear modulus for orthotropic materials presents another challenge since it must be determined experimentally. In the absence of any experimental data or an analogous analytical model, the approximate expression proposed by Szilard (Szilard, 1974) for the shear modulus of an orthotropic material was employed in the present study. This expression, developed mainly for the determination of the torsional rigidity of orthogonally stiffened plates, can be expressed as follows:

$$G_{orth} = \frac{\sqrt{E_1 E_2}}{2(1 + \sqrt{\nu_{12} \nu_{21}})} \quad (51)$$

In this study, masonry walls are modeled by quadrilateral thin shell elements. The element is divided into a number of isotropic and orthotropic masonry layers in the thickness direction. This layered representation of the element is shown in Fig. 3.14.

3.3 Numerical Implementation and Solution

The different constitutive relationships for masonry and reinforcement are implemented in a displacement-based finite element formulation. The nonlinear equations are solved in an incremental form where, for each load step, an iterative procedure was performed. The nonlinear equations of equilibrium were based on a dynamic formulation where the damping and mass matrix are set to a zero value and direct step-by-step time integration has been successfully applied. Then, the governing incremental equilibrium equation can be solved in a static fashion.

Only displacement-driven static response is considered in the current study. This has been so for two reasons: firstly, to capture the softening behaviour of the modeled system, and secondly, to simulate the available experimental results of different masonry elements that were tested under displacement-controlled conditions in an attempt to capture their softening behaviour experimentally.

Since one of the objectives of the numerical modeling of masonry shear walls is to trace the load/displacement relationship up to the ultimate stage, a new problem arose when the physical behaviour of laterally loaded reinforced concrete masonry shear walls indicated a softening behaviour. Snap-through and snap-back buckling phenomena have posed some of the most difficult problems in nonlinear structural analysis (Crisfield, 1981). Fig. 3.15 illustrates different dynamic snaps under displacement and load control solutions. Depending on the incrementing procedure (load or displacement), limiting points exist beyond which a conventional

solution may diverge. Different solution techniques were used to examine the most appropriate procedure for modeling the current structural element.

First, under load control procedure, it was found that if a constant load level incrementing procedure is chosen then the solution will diverge immediately when it reaches the limit point as shown in Fig. 3.16. To overcome this shortcoming, the modified arc length incrementing procedure was utilized. This procedure can be summarized as a load level reduction procedure to capture the softening behaviour beyond the limit point as can be seen in Fig. 3.17.

Second, under displacement control procedure, the softening behaviour can be captured by specifying a prescribed (known) displacement pattern and the equations of equilibrium are solved in a static fashion where the damping and mass matrix are set to a zero value. Also, the equations of equilibrium can be solved in a dynamic fashion where longer time steps are employed. This procedure will ensure a more stable solution due to the influence of inertia and damping which in turn will help in smoothing the solution and reduce the number of iterations.

Various methods of equilibrium iteration, Newton-Raphson, Modified Newton-Raphson, and BFGS methods, were applied and their effect on the stability of the solution was then observed. Also, different convergence criteria, including force, displacement, and energy, were utilized and their effects on the solution of highly nonlinear systems (such as cracked masonry panels) with softening behaviour were monitored.

3.4 Computer Implementation of Material Model

Fig. 3.18 illustrates the flow chart of the current model. First principal strains are calculated at time t , then stresses and incremental strains are calculated in the direction of principal strains. Next, the material is checked against failure in tension and compression. Based on the loading or unloading indicators, the incremental stresses are calculated based on an isotropic or orthotropic material matrix. Then, the total strains are calculated for time $t + \Delta t$. In case of compression failure,

appropriate stresses are released. In case of tension cracking, opening and closing of cracks are checked and the appropriate tension stiffening effect is modeled. Then, in case of equilibrium, the stiffness matrices are updated based on the loading or unloading cases and based on the material state: uncracked, cracked, crack closure or crushed.

3.5 Idealization of Masonry Shear Wall Structures

As mentioned earlier, the finite element method has been employed in this study to obtain an analytical model for reinforced masonry systems. Examination of the structural behaviour of different structural components of a masonry shear wall structure suggests that flat thin quadrilateral shell elements will best model its physical behavior.

In choosing the shell element, special care should be exercised to avoid the main sources of inaccuracy. Lower-ordered finite elements (such as constant strain elements) are known to perform poorly in such situations and experience what is called a *locking phenomenon* which gives rise to parasitic transverse shear stresses and results in spurious shear energy.

Several attempts have been made to overcome this spurious shear energy problem. One method of avoiding the inclusion of such spurious energies is by using a *uniform reduced integration scheme* (Zienkiewicz *et al.*, 1971) in which a lower order integration for all stress components is performed. A *selective integration scheme* was first suggested by Doherty *et al.* (Doherty *et al.*, 1969) and by Pawsey (Pawsey, 1971). In this scheme, the shear strain energy is evaluated by a lower-order integration than that used for other types of energies. This technique significantly reduces the *locking phenomenon* and also avoids activating the dangerous spurious modes associated with the *uniform reduced integration scheme*.

However, since convergence may not be assured with the use of the *selective integration schemes* (Wilson, 1971), their application has been limited to idealized geometries and isotropic materials. Wilson proposed another technique based on

the introduction of incompatible displacement modes in the basic quadrilateral element. Then, by minimizing the strain energy within the element, the magnitude of the unknown modes can be determined. Although this technique violates the inter-element compatibility, good results can still be achieved.

In the current study, the nature of the problem under investigation is highly nonlinear and the accuracy of the load-displacement history of a softening system is specifically targeted. Therefore, the suitability of any of the aforementioned techniques was examined using two cantilever beams with different geometries as well as load configurations and their responses to static loading were obtained. Results of the analyses are presented in Figs. 3.19 and 3.20 where they were compared with theoretical results using classical beam theory.

In the tables shown in Figs 3.19 and 3.20, **Quad4** is a standard isoparametric quadrilateral element, **Quad6** is a six-noded quadrilateral element in which the shear strains are integrated with (2×3) Gauss points, **Quad82** is an eight-noded isoparametric quadrilateral element in which the shear strains are integrated with a reduced scheme of a (2×2) Gauss points, **Quad83** is an eight-noded isoparametric quadrilateral element in which the shear strains are integrated with a (3×3) Gauss points, **SHELLN** (Algor, 1989) is composed of four linear strain triangular elements with reduced shear integration, and **Quad6k** (Kashif, 1992) is a six-noded quadrilateral element associated with incompatible displacement modes.

While best displacement results were obtained using **Quad83** and **Quad6k** elements, element **Quad83** illustrated a superiority regarding stress predictions which is considered to be a major advantage since the nature of the current problem is materially highly nonlinear. Therefore, the element **Quad83** was chosen for the current study. **Quad83** was developed by Bathe and Bolourchi (Bathe and Bolourchi, 1980) and was originally meant as a geometric and material nonlinear shell element.

3.6 Calibration of the Finite Element Models: Example No. 1

3.6.1 Experimental Data

To evaluate the capability of the thesis' smeared crack model, the in-plane resistance of two reinforced masonry shear wall panels tested by Shing *et al.* (Shing *et al.*, 1989) was considered. The wall specimens were 6-ft long and 6-ft high, and they were constructed of fully-grouted $6 \times 8 \times 16$ -in. hollow concrete blocks, having five standard reinforcing bars uniformly distributed in the vertical as well as horizontal directions at a centre-to-centre spacing of 16 inches. The lateral displacement was controlled by a horizontal actuator and consisted of symmetric cyclic reversals with gradually increasing amplitudes while the axial load was kept constant during each test.

Two test panels designated as specimens 5 and 12 in the aforementioned experimental study were utilized to calibrate the finite element smeared crack models. Both wall panels were subjected to a constant axial stress of 100 psi; specimen 12 failed by a ductile flexural mode that displayed severe yielding of the vertical reinforcing bars accompanied by considerable toe crushing. On the other hand, specimen 5 experienced a brittle shear failure dominated by diagonal tensile cracks.

3.6.2 Finite Element Model

A number of recent studies (e.g. Bazant, 1986; Bazant and Pijaudier-Cabot, 1987) revealed that mathematical modeling of softening damage necessitates a certain length parameter that is an additional property of the material and is named the characteristic length. Although the characteristic length has been used in various nonlocal models for strain softening, its value has so far been estimated only indirectly and probably with considerable uncertainty.

Depending on the type of the material model, the characteristic length can have two interpretations. First, if the softening damage is modeled by means of a strain-softening continuum, using localization limiter seems to be necessary to prevent the damage zone from localizing to a zero volume. Therefore, the characteristic length is defined to determine the size of a point neighborhood over which certain strain

variables are determined by averaging over that neighborhood. Without the aforementioned characteristic length, the strain softening models for damage can correctly describe only the situations where the damage remains distributed (nonlocalized damage). Second, the discrete crack models with a softening cohesion zone at the crack front (Hillerborg *et al.*, 1976) can correctly describe localized damage but may yield ambiguity for situations where cracking remains distributed such as the case with reinforced concrete/masonry where reinforcement adds stability for the damaged systems. Therefore, the discrete crack models must also contain a material length limiter that indicates the minimum possible spacing of cracks.

Bazant and Pijaudier (Bazant and Pijaudier, 1989) introduced methods for determining the characteristic length of a nonlocal continuum model to model tension-softening problems. However, no measurement procedures have been suggested to measure the characteristic length of a nonlocal continuum with compression-softening. On the other hand, since softening characteristics obtained from any material tests merely reflect the overall behaviour of the test specimen rather than the true material properties (such as strain localization and crack band), calibration of the material softening parameters remains a major challenge. Adding to the above challenge, the typical difficulties such as solution strategy and convergence criteria as well as boundary conditions can cause strain localization in numerical modeling. Besides, in an actual specimen test, typical difficulties that cause strain localization such as material imperfections and loading conditions may have a significant influence on the softening behavior of the material. Therefore, this softening behaviour may be unique to that specific test procedure.

Since the objective of the current numerical study is to avoid the increase in the computational effort that will result if one decides to use discrete-crack models which again may not be feasible for diffuse crack problems, the decision was then taken to use smeared crack models to allow future modeling of full scale structures taking into account a rational localization limiter.

Now, it is well established that strain-softening exists only in a heterogeneous

material where failure occurs by unstable localization of strain in which the stored strain energy is transferred into a strain-softening region, whose size (in concrete and masonry) is dependent on aggregate size, spacing of reinforcement and element geometry (Bazant, 1976). The aforementioned concept is also valid within the finite element analyses of reinforced masonry structures where prediction of limit loads depends strongly on the chosen size of finite elements. By means of a stability analysis, a lower limit on the size of the softening region that permits ductility can be predicted.

In the context of spurious mesh-size sensitivity, strain localization can be treated by either using a limiter in the model to prevent it or by modifying the softening parameters accordingly to maintain the equality of energy dissipation with respect to different meshes. In reinforced concrete/masonry, while other considerations rather than size limiter and equity of energy dissipation concept have to be included when modeling the tension-stiffening effect of reinforcing steel, the localization of tensile deformation can be greatly prevented as a result of the stabilization effect of the steel reinforcement (Bazant and Pijaudier-Cabot, 1987). Therefore, in the current study the softening branch of the tensile stress-strain curve was kept independent from the element size or the fracture energy.

On the other hand, since adjusting the softening parameters is more complicated in the case of compressive fracture because of the lack of uniquely defined compressive fracture behaviour, using the deformation localization size limiter strategy seems to be more convenient. Bazant (Bazant, 1976) recommended that the proper size must be determined experimentally. Therefore, the finite element size was chosen to be equal to the experimentally observed dimension of the compressive fracture zone at the base of wall panels. This procedure is confirmed by the analytical modeling of shear walls carried out by Lotfi and Shing (Lotfi and Shing, 1990) and Seible *et al.* (Seible *et al.*, 1989).

The finite element model of the wall panels is shown in Fig. 3.21. The analysis is conducted under displacement control by specifying horizontal displacement

increments at the top nodes as shown in Fig. 3.21a, while a constant axial load is applied at the beginning of the incremental analysis. Only monotonic displacement patterns are considered in the numerical analyses. Eight node quadrilaterals with 3×3 Gauss points are used in the current analysis. The material properties utilized in the numerical analyses were obtained from the test data published by Shing *et al.* (Shing *et al.*, 1989 *et al.*, 1989) and are shown in Fig. 3.21b.

3.6.3 Numerical Results

The results for specimen 12 are shown in Fig. 3.22. As shown in Fig. 3.22a, the numerical load-displacement curve matches well the experimental envelope curve. Furthermore, the vertical steel yields substantially due to the widening of flexural cracks, while no yielding is detected in the horizontal steel. This can be seen in Figs. 3.22b and 3.22c. Toe crushing first occurs at a lateral load of 68 kips in the analysis versus 71 kips in the actual test. The extent of the crushing is very similar to that observed experimentally.

The same panel was modeled numerically using the plasticity J_2 model developed by Lotfi and Shing (Lotfi and Shing, 1991) and the load-displacement curve, crack pattern, and yield pattern are shown in Figs 3.22a, 3.22c, and 3.22e, respectively. By comparing Figs. 3.22b and 3.22c, and comparing Figs. 3.22d and 3.22e, almost complete agreement between the current model and the plasticity model was found.

The results for specimen 5 are shown in Fig. 3.23. As shown in Fig. 3.23a, the numerical load-displacement curve agrees in general with the experimental envelope curve. The predicted ultimate load by the analysis is slightly higher than that shown in the test and occurs at a lower lateral displacement. Again, the same panel was modeled numerically using the plasticity J_2 model developed by Lotfi and Shing (Lotfi and Shing, 1991) and the load-displacement relationship is shown in Figs 3.23a. In both models, the lateral resistance continues to increase as the tensile strain at the diagonal cracks increases. While the plasticity model failed to capture the rapid drop in the lateral resistance once major diagonal cracks start opening, the current model successfully captured this drop in the lateral resistance as a diagonal crack

opened and consequently the aggregate interlock forces and the residual shear at the compression toe were considerably reduced. This rapid drop in the lateral resistance is due to the small amount of shear reinforcement as shown by the experimental envelope curve given in Fig. 3.23a and confirmed by the yield pattern shown in Fig. 3.23d.

By comparing Figs. 3.23b and 3.23c, and also by comparing Figs. 3.23d and 3.23e, almost complete agreement between the current model and the plasticity model in detecting the crack and yielding patterns was found. Also, it was observed from the numerical results that the post-peak load degradation in panel 5 is accompanied by compression toe crushing and substantial flexural cracks.

3.6.4 Parametric Study

The problem encountered in modeling the shear dominant panel 5 required further study in which different smeared crack approaches, different shear retention factors, and different tension stiffening models are considered. Figs. 3.24 and 3.25 illustrate the influence of the current rotating crack model and fixed crack model on the load-displacement envelope for specimens 12 and 5, respectively; analytical results were compared with the experimental envelopes. Also, these results were compared with the rotating crack model developed by Seible *et al.* (Seible *et al.*, 1989). All the results were based on compressive and tensile constitutive laws developed by Vecchio and Collins and on the shear modulus G given in Eq. (3.41) where the shear retention factor η_s was assumed as 0.5 for all cases.

The results, shown in Fig. 3.24 for specimen 12, illustrate good agreement between the current rotating crack model and the experimental results for shear walls dominated by a flexural failure mode up to the ultimate stage of loading; however, the fixed crack model results indicate some discrepancies and cause early termination of the lateral load capacity.

The results, illustrated in Fig. 3.25 for specimen 5, indicate that while almost all the rotating crack models showed general agreement with the reported experimental results up to 70 percent of the ultimate load in the pre-peak stage, the fixed crack

model failed to match the load-displacement envelope up to this limit. On the other hand, the descending branch of the load-displacement envelope indicated dependency on the model where the Seible *et al.* results indicated an unexplained lateral residual resistance of the masonry wall and the fixed crack model showed a sudden sharp failure after reaching the ultimate load which was due to the instability of the model in capturing the softening of the system.

The influence of different shear retention formulas with the current rotating crack model was assessed. Three models were considered: the Darwin and Pecknold model, the coaxial rotating crack model (Eq. 3.43), and finally the non-coaxial rotating crack model where the shear retention factor was represented by Eq. 3.42 (Rots and Blaauwendraad, 1989). As illustrated in Fig. 3.26, the influence of the shear retention was only observed in the post-peak region of the load-displacement curve (the descending part). It was observed that the greatest strength degradation is given by the coaxial rotating crack model, while, for the non-coaxial rotating models, the shear retention causes the principal stresses to rotate with respect to the axes of orthotropy. The larger the shear retention is, the more significant is the rotation, which therefore results in higher principal stresses and thereby a higher load resistance.

Fig. 3.27 illustrates the influence of different tension-stiffening models on the load-displacement envelope. This effect was studied with the rotating crack model that employs Vecchio and Collins compressive constitutive law and a shear retention factor (η_s) of 0.5. Generally, it can be observed that the influence of the material tension-stiffening on the load-displacement is more significant in the ascending part of the envelope (pre-peak region) than on the descending part of the envelope (the post-peak region). No preference was given to any of the employed tensile-stiffening models. Also, regardless of the small differences between the predicted ultimate strengths that correspond to various tension-stiffening models, the final failure modes were the same and the descending curve branches almost coincided.

Figs 3.28 and 3.29 illustrate the influence of various compressive stress-strain relationships of masonry on the load-displacement history of the same two walls tested

by Shing and designated 12 and 5 (flexure and shear dominated behaviour, respectively). Four models were employed and results were compared with the experimental findings. In all the numerical analyses presented in Figs. 3.28 and 3.29, a smeared rotating crack model was employed along with the tension-stiffening model by Vecchio and Collins; shear retention parameters were determined based on a coaxial rotating crack concept.

As illustrated in Fig. 3.28, for the flexure dominated wall (wall 12) all the models matched well the experimental load-displacement relationship up to the peak load and also closely matched experimental behaviour beyond this peak point up to the ultimate failure with one exception; the parabolic relationship introduced a premature failure after reaching the peak load.

For the shear dominated wall (wall 5) as illustrated in Fig. 3.29, all the models matched well the experimental load-displacement relationship up to 70% of the peak load. Beyond this point, only Bathe's model matched well the experimental load-displacement relationship up to the peak load.

3.6.5 Discussion of Results

The analysis of the flexure dominated wall indicated a very good match with the experimental results. On the other hand, the analysis of the shear dominated wall indicated a slight increase in the wall lateral resistance over the experimental results; a sudden drop was experienced once major diagonal crack openings occurred. As a diagonal crack opened, the aggregate-interlock forces and the residual shear at the crack interface were substantially reduced. Because of the small amount ρ_v shear reinforcement, this led to a rapid drop of the lateral load resistance. This, to a great extent, was predicted by the current model. Also, it is observed from the numerical results that the post-peak ultimate load degradation is accompanied by a diagonal strut mechanism which failed quickly by crushing of the concrete in the diagonal strut direction.

Lotfi and Shing (Lotfi and Shing, 1991) analyzed the same shear wall (specimen 5) using a plasticity J_2 model. The numerical solution of the aforementioned shear wall

was influenced by spurious shear strength which consequently prevented the realistic modeling of brittle behaviour. They claimed that this spurious lateral resistance is due to the diagonal compressive strain that is associated with shear distortion which in turn is caused by the diagonal crack opening. The aforementioned phenomenon can be best explained by a single element subjected to pure shear conditions. The corresponding Mohr's circles of stresses at a single Gauss point are shown in Fig. 3.30. Once a diagonal crack has developed and the diagonal tensile strength is being reduced to zero, Mohr's circle of stress will be shifted to the position shown in Fig. 3.30(b).

Lotfi and Shing claimed that the shear capacity will be maintained as long as the diagonal compressive strength remains, regardless of the shear retention along the diagonal crack surface. While the last statement is true to a great extent, it could not explain the spurious shear strength. Utilizing the thesis' current model indicated the development of the strut mechanism. Concurrently, due to the increased compressive strains, a progressive diagonal compressive failure was initiated starting from the crushed toe and extending diagonally until complete failure of the model. Numerical results, using the current model, indicated no spurious shear strength for the modeled shear dominated specimen 5 and the sudden drop due to the excessive opening of the diagonal crack has been properly captured. This represents an advantage over the plasticity model.

The parametric study of the effect of three different tension-softening models indicates that the slope of the tension-stiffening of the material model has little influence on the global behaviour of the shear-dominated wall panel and focuses only on the pre-peak region; hence no preference was given to one of the models over the others.

On the other hand, the influence of the shear retention factor on the shear-dominated wall panel has focused only on the post-peak region where it affects the rate of degradation of the panel's lateral load resistance. Lotfi and Shing have claimed that the shear retention influence appears to be indirect and artificial and due entirely

to the rotation of axes of orthotropy.

The parametric study of the influence of various compressive stress-strain relationships of masonry on the load-displacement history of shear walls indicates that the compressive constitutive law may have a slight effect on the load-deformational characteristics of shear walls failing in flexure where reinforcing bars experience extensive tensile yielding. On the other hand with walls that fail in a shear brittle fashion, only Bathe's model indicates a very good match with the experimental results especially in the stage above 70% of the peak load. This is due to the fact that the aforementioned model provides a smaller value for the tangential stiffness at a given strain and this in turn makes the analytical model more flexible under the applied loading conditions.

3.7 Calibration of the Finite Element Models: Example No. 2

3.7.1 Experimental Data

To evaluate the capability of smeared crack models in modeling three-dimensional problems such as the case of intersecting masonry shear walls in a structural building system, one (designated F1) of four flanged masonry walls tested by Priestley and Limin (Priestley and Limin, 1990) was modeled. The wall specimen was of 12-ft (3660mm) height, 8'-8" (1640mm) flange width, and 3'-10" (1170mm) web length as shown in Fig 3.31. The wall was constructed from nominal 6-in. (150mm) concrete masonry units, using open-end bond-beam units throughout the body of the wall and half-lintel units, or open-end standard units with the central web knocked out to mid-height at the wall edges. Monolithic action of the flange and the web was provided by cutting out the face shells of the inner side of the flange units on the web line to allow full penetration of the grout along the height of the flange-web interface. Seven #6 grade 60 reinforcing bars in the flange and three #6 grade 60 bars in the web at 16 in. (400 mm) nominal spacings were used to reinforce the flanged wall. Transverse reinforcement of #4 grade 60 bars were placed in the flanged wall at 8-in. centers over the full height of the wall. The lateral displacement was controlled by a horizontal actuator; the wall was subjected to symmetric cyclic reversals with

gradually increasing amplitudes, while the axial load was kept constant at 100 psi axial stress during the test.

3.7.2 Finite Element Model

Based on the discussion and literature review presented in Section 3.6.2 regarding the strain localization and characteristic length, the finite element size was chosen to be equal to the experimentally observed dimension of the compressive fracture zone.

The finite element model of the wall panels is shown in Fig. 3.31. The analysis was conducted under displacement control by specifying horizontal displacement increments at the top node while a constant axial load is applied at the beginning of the incremental analysis. Only monotonic displacements were considered in the numerical analysis of this example. Eight node quadrilaterals with 3×3 Gauss points were used. The material properties utilized in the numerical analyses were obtained from the test data published by Priestley and Limin (Priestley and Limin, 1990) and are shown in Fig. 3.31.

3.7.3 Numerical Results

The numerical analyses results for the flanged wall are compared with the experimental results in Fig. 3.32. As illustrated in Fig. 3.32, the numerical load-displacement curve matches well the experimental envelope curve. Furthermore, the vertical steel yields substantially due to the widening of flexural cracks, while no yielding is detected in the horizontal steel. Results can be classified into two sets depending on the direction of the load application which controls whether the web or the flange is in tension or in compression. When the web is under compression, analytical results indicated that first yield occurred at a lateral load of 47 kips and toe crushing first occurred at a lateral load of 51 kips. Unfortunately, similar experimental results are not available for comparison. Also, the maximum shear load of 60.3 kips was predicted numerically at a displacement of 1.1 in. versus 63.4 kips at 1.6 in. lateral displacement recorded experimentally. When the web is in tension, a good match between the numerical and experimental load-displacement relationships was found.

Fig. 3.1 Increase of strength parameters for masonry model under multiaxial conditions (Bathe and Ramaswamy, 1979).

Fig. 3.2 Stress-strain relationship for cracked masonry in compression (Vecchio and Collins, 1986).

Fig. 3.3 Idealized stress-strain curves for confined and unconfined concrete masonry.

Table 3.1 Various parameters involved in the curve fitting model by Hart et al..

Type of Confinement	Ascending Branch	Descending Branch	
	C	D	E
URM	-146 900	0.110	- 613.8
UBC	-175 400	0.201	- 516.2

Fig. 3.4 Cyclic compressive stress-strain relationship for masonry (Reinhardt et al.).

(a)

(b)

(c)

(d)

(e)

(f)

Fig. 3.5 Comparison of some of the existing tension-softening models.

Fig. 3.6 Biaxial Tension-stiffening model based on Stevens et al. (Stevens, Uzumeri and Collins, 1987).

Fig. 3.7 Proposed Tension-stiffening model based on Gopalaratnam and Shah test results and crack band theory (Bazant and Oh, 1983).

Fig. 3.8 Proposed cyclic tension-stiffening model for masonry based on Gopalaratnam and Shah test results and crack band theory (Bazant and Oh, 1983).

Fig. 3.9 Elastic-plastic response of reinforcing steel.

(a) Stresses Applied to Cracked Element

(b) Calculated Average Stresses

(c) Local Stresses at a Crack

Fig. 3.10 Comparison of local stresses at a crack with calculated average stresses.

Fig. 3.11 Transmitting shear stresses across a crack by aggregate interlock.

Fig. 3.12 Shear retention factor of Eq. 3.42.

Fig. 3.13 Orthotropy of concrete block masonry.

Fig. 3.14 Layered composition of thin shell element.

Fig. 3.15 Snap buckling.

Fig. 3.16 Constant load level incrementing procedure.

Fig. 3.17 Modified arc length incrementing procedure with Modified Newton-Raphson technique.

Fig. 3.18 Flow chart of the model.

Material Properties:

Modulus of Elasticity $E = 100\,000$

Poisson's Ratio $\nu = 0.3$

Thickness $t = 1.0$

Mass Density $\rho = 0.1$

Element Type	Loading Case 1		Loading Case 2	
	Displacement at Point A	Stress at Point B	Displacement at Point A	Stress at Point B
Theory	0.135	90.0	0.202	90.0
Quad4	0.069	47.8	0.103	50.4
Quad6	0.123	90.0	0.184	90.0
Quad82	0.134	90.0	0.202	90.0
Quad83	0.134	90.0	0.202	90.7
SHELLN	0.116	62.7	0.167	65.9
Quad6k	0.135	85.5	0.202	90.0

Fig. 3.19 Static analysis results for cantilever beam No. I.

Loading Case 1 **Loading Case 2** **Material Properties:**

Modulus of Elasticity $E = 100\,000$

Poisson's Ratio $\nu = 0.3$

Thickness $t = 1.0$

Mass Density $\rho = 0.1$

Element Type	Loading Case 1		Loading Case 2	
	Displacement at Point A	Stresses at Point B	Displacement at Point A	Stresses at Point B
Theory	1.000	16650.0	1.000	11112.0
Quad4	0.720	10280.0	0.674	8231.0
Quad6	0.968	16650.0	0.910	11110.0
Quad82	1.024	11680.0	0.989	11110.0
Quad83	1.030	16539.0	0.989	11155.0
SHELLN	0.857	16654.0	0.913	11117.0
Quad6k	1.035	13937.0	1.000	11112.0

Fig. 3.20 Static analysis results for cantilever beam No. II.

(a) Test specimen and finite element discretization

Test Wall No.	f'_m (psi)	A_s , vertical		A_s , horizontal		Additional Vertical Load (psi)	Failure Mode
		ρ_s (%)	f_y (ksi)	ρ_s (%)	f_y (ksi)		
12	3 200	0.38	63	0.24	67	100	Flexure
5	3 000	0.74	70	0.14	56	100	Shear

$$f'_c = 100 \text{ psi}$$

$$\text{Strain hardening in steel } \alpha = 0.005$$

$$\epsilon_p = 0.0026$$

(b) Material properties

FIG. 3.21 Single storey wall model and material properties.

Fig. 3.22 Hypoelastic versus plasticity model for wall 12, flexure dominant failure.
 (a) Effect on the load-displacement history,
 (b) and (c) Cracking pattern (at 0.6 in. displacement),
 (d) and (e) Yielding and crushing patterns (at 0.6 in. displacement).

Fig. 3.23 Hypoelastic versus plasticity model for wall 5, shear dominant failure.
 (a) Effect on the load-displacement history,
 (b) and (c) Cracking pattern (at 0.48 in. displacement),
 (d) and (e) Yielding and crushing patterns (at 0.48 in. displacement).

Fig. 3.24 Influence of rotating and fixed crack models on the load displacement history. (wall 12, flexure dominant failure)

Fig. 3.25 Influence of rotating and fixed crack models on the load displacement history. (wall 5, shear dominant failure)

Fig. 3.26 Influence of shear retention with rotating crack model on the load displacement history. (wall 5, shear dominant failure)

Fig. 3.27 Influence of tension softening with Non-coaxial rotating crack model on the load displacement history. (wall 5, shear dominant failure)

**Fig. 3.28 Influence of various compressive stress-strain relationships on the load displacement history.
(wall 12, flexure dominant failure)**

**Fig. 3.29 Influence of various compressive stress-strain relationships on the load displacement history.
(wall 5, shear dominant failure)**

Fig. 3.30 Spurious shear strength.

(a) Test specimen and finite element discretization

Wall Element	f'_m (psi)	A_s , vertical		A_s , horizontal		Additional Vertical Load (psi)
		ρ_s (%)	f_y (ksi)	ρ_s (%)	f_y (ksi)	
Web	2 880	0.65	68	0.44	76	100
Flange		0.49	68	0.22	76	

$f_{cr} = 320$ psi $\epsilon_o = 0.0026$ Strain hardening in steel $\alpha = 0.005$

(b) Material properties

Fig. 3.31 Flanged wall model and material properties.

Fig.3.32 Flanged wall model: comparison of analytical and experimental results (Priestley and Limin, 1990).

Chapter 4

EXPERIMENTAL PROGRAM

4.1 General

This chapter deals with the experimental phase of the research where the objectives behind the experimental work will be summarized. Also, numbers and dimensions of the wall panels, fabrication procedure, instrumentation, test setup, and type of loading will be presented. Materials properties including unit and assemblage testing results will also be reported.

4.2 Scope and Objectives

Reinforced masonry shear walls typically form the lateral seismic load resistance elements in multi-story masonry buildings. The design of shear walls for all critical seismic limit states requires a detailed understanding of the complete reversed cyclic response of reinforced concrete/masonry walls to establish the behavioural characteristics of the assemblage of masonry walls under seismic excitation.

To investigate the complete nonlinear response of structural components under seismic loading, increasing cyclic quasi-static experiments are frequently performed. Data from full-scale experimental studies on masonry shear walls are necessary to establish behavioural characteristics of masonry assemblages under cyclic loading at cracking, yielding, crushing, peak strength and deformation limit states before comprehensive and representative design models can be developed.

In actual masonry buildings, shear walls are usually subjected to simultaneous in-plane lateral and vertical loads during seismic excitation. Depending on geometry, loading conditions and design, two distinct behavioural failure mechanisms have been observed with shear walls subjected to lateral in-plane loads. The first is the flexural mechanism in which the wall is dominated by a ductile failure accompanied by tensile yielding of vertical reinforcement and compressive crushing of masonry. The second is the shear mechanism in which wall panels fail in a predominantly shear mode in a more brittle fashion than those dominated by flexural yielding.

While simple or complex models can accurately evaluate the flexural strength of a reinforced masonry panel, no accurate theoretical model has been developed to predict the shear strength of a reinforced masonry panel dominated by diagonal tensile cracking. The shear strength of a reinforced masonry wall depends on the cracking load along with many other mechanisms. This cracking load is governed by many factors such as the tensile strength of masonry, the applied axial vertical stress level, and the wall aspect ratio. Also, after cracking, the shear strength of a reinforced masonry wall depends on several other mechanisms, such as the aggregate-interlock mechanism and the dowel action of the vertical steel. Note that Canadian shear wall design practice has relied on a working stress approach for over thirty years (CSA-S304-M78 and CSA-S304-M84) and only the very recent code developments (CAN-S304-M95) take into account the ultimate load limit states approach.

To develop a new model for predicting the shear strength of a reinforced ma

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sonry panel exhibiting shear brittle failure, certain mechanisms need to be modeled and the extension of the effect of certain parameters needs to be investigated. In the present research, while various mechanisms were modeled and have been presented in previous chapters, these models need to be verified experimentally by examining reinforced masonry shear walls of differing geometry, loading conditions and design. Therefore, in the current experimental investigation, the effects of aspect ratio, applied axial stress and amount of vertical reinforcement on the residual lateral strength of reinforced masonry shear walls were investigated experimentally.

4.3 Test Specimens

A total of five 1400 mm high reinforced masonry walls were subjected to lateral cyclic reversed displacement patterns simulating earthquake excitation while a constant pre-compressive vertical load was applied to simulate the dead load of the structure. Each wall was built of $140 \times 190 \times 400$ mm hollow concrete blocks in single wythe construction as shown in Fig. 4.1. Summary of test walls including geometry, reinforcement ratios and loading conditions is illustrated in Table 4.1.

All the walls were designed to experience brittle shear failure modes accompanied by diagonal cracking. The first three specimens had aspect ratios (height/length) of 1.0, 0.636 and 0.467 respectively. These three wall panels had the same vertical and horizontal reinforcement percentages of 0.4 % and 0.2 % respectively, and were tested under a constant applied vertical compressive stress of 0.69 MPa (100 psi) as shown in Table 4.1. Test results of these three specimens (designated walls 1, 2 and 3) were used to study the effect of aspect ratio on the shear strengths of reinforced masonry shear walls.

In studying the effect of the amount of vertical reinforcement on the lateral shear resistance of reinforced masonry panels, a reinforced masonry wall panel, designated wall 4, with an aspect ratio of 0.636 was tested under a vertical stress of 0.69 MPa (100 psi). The wall had vertical and horizontal reinforcement ratios of 0.6% and 0.2%, respectively.

Also, to investigate the effect of the applied compressive stress on the lateral resistance of masonry panels, a reinforced masonry wall panel, designated wall 5, with an aspect ratio of 0.636 was tested under a vertical stress of 1.73 MPa (250 psi). The wall had a vertical and horizontal reinforcement ratios of 0.4 % and 0.2%, respectively.

The reinforcement ratios were chosen to ensure a shear dominated failure mode. All walls were modeled using the newly developed nonlinear finite element model (presented in the early chapters of this thesis) to predict their modes of failure. All walls were fully grouted, with vertical and horizontal reinforcement spaced uniformly at 400 mm. The horizontal reinforcing bars were placed at the middle height of course by knocking off the cross webs to half-height of the block. The horizontal reinforcement has 180° hooks around the extreme vertical steel at both far ends.

Each specimen had heavy reinforced concrete top and bottom blocks. Bottom blocks, of 300 × 1100 mm cross-section, were used to provide fixity at each wall base and transfer this fixity to the strong floor of the structural laboratory using six 1830 mm (6 ft) long 57 mm (2.25 in.) diameter high strength tie rods. A rectangular heavily reinforced top block, of 400 × 500 mm cross-section, was cast on the top of each masonry wall to simulate the actual floor-slab system and to ensure a uniform vertical and horizontal load transfer to the top of each masonry wall. The vertical reinforcement ran continuously from the base block to the top block and was properly anchored. All reinforcing bars were of Grade 400.

4.4 Fabrication

The five test specimens were constructed in several stages. The base block was cast first with the vertical wall reinforcement protruding from the base. Secondly, the walls were constructed by experienced masons and were grouted two days after construction with a ready-mixed grout that had an workable slump of at least 250 mm. Finally, the top concrete blocks were cast. Note that all stages

of construction were performed at the same time for all the walls and by the same experienced masons to ensure consistency in workmanship. All the walls were tested approximately six months after construction. The formwork for the top and bottom blocks was made from plywood reinforced with 50×100 mm wooden studs. The plywood sheets were lightly coated with form oil as bond breaker and the concrete was properly vibrated using an electrical vibrator. Prior to casting, four plastic tubes were placed in the top block to allow the insertion of four pulling bars to be connected to the hydraulic actuator. Various fabrication and construction stages are illustrated in Figs. 4.2 to 4.5. While the test walls were left to cure under laboratory conditions, all material properties specimens (mortar cubes, grout cylinders, ungrouted and grouted prisms) were kept moist by means of polyethylene wrapping for the first seven days of curing. After seven days, the specimens were uncovered to cure under laboratory conditions until testing time; specimens were tested at about the same time as the shear walls. Laboratory conditions consisted of a relative humidity of about 25% and an air temperature of about 20°C.

4.5 Material Properties

The masonry walls were built with $140 \times 190 \times 400$ mm hollow concrete blocks of nominal compressive strength equal to 15 MPa and type S mortar. The ready-mixed grout mix consisted of 1:4 portions of Portland cement and fine aggregate by volume. Materials tests were conducted for each specimen in accordance with applicable CSA and ASTM standards. The compressive strength of mortar was obtained from 50 mm mortar cubes. Three mortar cubes were prepared during the construction of each wall to provide a total of 15 mortar cubes. Grout strength was obtained from specimens prepared by arranging blocks as illustrated in Fig. 4.6 and covering the inside of the masonry units with absorbent paper towelling and then pouring the grout into the $90 \times 90 \times 190$ mm space. Also, five cylinders were prepared for testing the compressive strength of the grout to compare absorbent versus non-absorbent compressive strengths. The compressive strength of

masonry prisms was obtained by testing five fully grouted, two-course high prisms; results were compared with those obtained by testing another set of five ungrouted specimens.

The construction of the masonry prisms is illustrated in Fig. 4.7. The deformational behaviour of grout prisms, grout cylinders, masonry units and masonry prisms was monitored using LVDT's and Demec gauges as shown in Figs. 4.8 to 4.11. Stress-strain relationships for grout, masonry units and masonry prisms are plotted in Fig. 4.12. The mortar, grout, masonry unit and masonry prisms average compressive strengths obtained from the material testing are summarized in Table 4.2.

All deformed reinforcing bars 10M, 15M and 20M were of grade 400 steel. Tensile coupons were tested under monotonic loading to determine the stress-strain characteristics of the steel. Strain gauges and a 50 mm extensometer were used to obtain the stress-strain relationships illustrated in Fig. 4.13. Other reinforcing steel properties are summarized in Table 4.3.

4.6 Instrumentation

Readings of the applied vertical and horizontal loads as well as deformations at the key points of the specimens were recorded. Linear Variable Differential Transducers (LVDT's) were used to monitor the lateral deflection, flexural deformation, shear deformation, base slip, and base uplift of each specimen. All deflection measurements using LVDT's were measured with respect to the base block so that sliding of the specimen on the floor during loading would not affect the readings.

As indicated in Fig. 4.14, electrical resistance strain gauges were placed on the vertical reinforcement within the most highly stressed region at the joint between the wall and the footing to monitor the first yield as well as the distribution of the tensile strain in the vertical steel bars. Seventeen LVDT's were used to monitor the wall's deformational behaviour as shown in Figs. 4.15 to 4.19. LVDT's 1 and 2 were used to measure the lateral displacement at the top of the wall. LVDT's 3 and 4 used to record the wall vertical displacements were mounted over the top of

the wall at its extreme ends. LVDT's 5 and 6 were used to monitor the wall overall shear deformation. LVDT's 7, 8 and 9 were employed to monitor the slippage at the wall base interface with the concrete bottom block. LVDT's 10, 11, 12 and 13 were utilized to record the masonry vertical deformation along the wall length. LVDT's 14 and 15 were used to measure the masonry wall deformations along lines that make a 45° angle with the wall edge. LVDT's 16 and 17 were employed to monitor the opening of the interface between the bottom of the masonry wall and the concrete base.

4.7 Test Setup

The test setup is shown in Figs. 4.20 and 4.21. A displacement controlled 667 kN (150 kips) hydraulic actuator was used to apply lateral loads to the shear walls. Vertical loads were applied by means of four Dywidag bars anchored underneath the strong floor and stressed against the top concrete block of each specimen through stiff coil springs. The flexibility of the springs ensured that the compressive load remained relatively constant as the specimen displaced laterally and vertically. The applied axial load was chosen to provide a uniform stress level of 0.69 MPa (100 psi) including the self-weight of the walls and the weight of the top concrete block at the base of the wall. The vertical pressure was applied using an electric pump connected to hydraulic jacks mounted over the top of the concrete block. For walls no. 1, 2, 4 and 5, two 10-ton jacks were used at the top of each wall while, for wall no. 3 (the longest wall), four 10-ton jacks were employed to ensure a uniformly distributed vertical stress. The electric pump with a self adjusted hydraulic pressure system provided a constant vertical pressure during testing. The electric pump attachments and the vertical load application units can be seen in Figs. 4.22 to 4.24.

To minimize any out-of-plane wall displacements during lateral loading, a guiding mechanism consisting of four casters mounted horizontally on a supporting rigid frame was utilized. This guiding mechanism is illustrated in Figs. 4.23 and 4.24.

4.8 Load/Displacement Sequence

Each specimen was subjected to a prescribed lateral displacement history under a constant axial load. A standard lateral displacement history similar to the one shown in Fig. 4.25 was employed. The displacement history consisted of sequences of fully reversed displacement cycles. The displacement amplitude of the first cycle of each sequence was identical to the maximum amplitude of the previous sequence. It was then followed by three cycles of an increased amplitude, and finally by one cycle of decaying amplitude. The decay cycle had an amplitude of one-half of the maximum amplitude of the sequence. The sequence was repeated, with an amplitude increment each time, until the specimen lost 50% of its maximum load resistance or the displacement limit of the horizontal actuator was reached. The application of the lateral displacement was very slow (0.2 - 0.8 mm/min): each wall took 3 to 4 days to reach ultimate failure. The aforementioned load sequence was chosen based on a literature review of various experimental studies carried out on reinforced concrete and masonry shear walls. Fig. 4.26 shows the load/displacement control and data acquisition units. Also, Fig. 4.27 represents a photo of the computer screen during testing; it indicates various features of the control unit including graphic illustration of the load-displacement relationship and various strain gauge and LVDT readings.

Table 4.1 Masonry wall specimens

Test Wall No.	Wall Aspect Ratio	A_s , vertical ρ_v (%)		A_s , horizontal ρ_h (%)		Additional Vertical Stress (MPa)
1	1.000	4# 15 M	0.4	4# 10 M	0.2	0.69 (100 psi)
2	0.636	6# 15 M	0.4	4# 10 M	0.2	0.69 (100 psi)
3	0.467	8# 15 M	0.4	4# 10 M	0.2	0.69 (100 psi)
4	0.636	6# 20 M	0.6	4# 10 M	0.2	0.69 (100 psi)
5	0.636	6# 15 M	0.4	4# 10 M	0.2	1.73 (250 psi)

Table 4.2 Average compressive strength for masonry materials

Masonry Units (MPa)	Mortar Cubes (MPa)	Grout Cylinders (MPa)	Grout Prisms (MPa)	UngROUTED Masonry Prisms (MPa)	Grouted Masonry Prisms (MPa)
23.5	14.7	17.9	21.3	15.9	12.1

Table 4.3: Average tensile strength for reinforcing steel

Bar Size	Yield Strength (MPa)	Ultimate Strength (MPa)
10 M	436	654
15 M	440	688
20 M	438	634

Fig. 4.1 Design of test specimens.

Fig. 4.2 Formwork for the concrete block bases.

Fig. 4.3 Construction of masonry walls.

Fig. 4.4 Grouting of shear walls.

Fig. 4.5 Formwork of the top concrete blocks.

Fig. 4.6 Preparation of grout prisms.

Fig. 4.7 Construction of masonry prisms.

Fig. 4.8 Testing of a grout cylinder.

Fig. 4.9 Testing of a grout prism.

Fig. 4.10 Testing of a masonry unit.

Fig. 4.11 Testing of a masonry prism.

Fig. 4.12 Stress-strain relationships for masonry units, grout and masonry.

Fig. 4.13 Stress-strain relationships of reinforcing steel.

Fig. 4.14 Location of strain gauges.

Fig. 4.15 Arrangement of LVDT's.

Fig. 4.16 General layout of attachments of LVDT's.

Fig. 4.17 LVDT's 1 and 2 controlling the wall lateral displacement.

Fig. 4.18 LVDT's 3 and 4 measuring the wall vertical displacements at both ends of the wall.

Fig. 4.19 LVDT's 16 and 17 measuring the opening of the interface between the wall and the base block.

Fig. 4.20 Loading frame.

Fig. 4.21 Test setup.

Fig. 4.22 Electric hydraulic pump and vertical load attachments.

Fig. 4.23 Wall guiding mechanism.

Fig. 4.24 Vertical load application components.

Typical Sequence of
Cycles

Fig. 4.25 Displacement history.

Fig. 4.26 Load control and data acquisition units.

Fig. 4.27 Photo of the control unit computer screen during testing.

Chapter 5

EXPERIMENTAL RESULTS OF TEST WALLS

5.1 General

This chapter summarizes the test results for the five test walls. The deformational characteristics, yield strength, diagonal cracking loads and crack patterns for the walls are presented. Observed failure modes and mechanisms are also discussed. Key wall test results have been summarized in Table 5.1.

5.2 Wall 1

As mentioned earlier, wall 1 had an aspect ratio of 1.0 and vertical and horizontal reinforcement ratios of 0.4% and 0.2%, respectively. The wall was subjected to the displacement history presented in Fig. 4.25 while a constant stress of 0.69 MPa (100 psi) was applied vertically. Fig. 5.1 shows the load-top displacement hysteresis curves for wall 1. The lateral displacement was monitored by LVDT 1 while the base slip was monitored by LVDT's 6, 7 and 8. In the relationships illustrated in Fig. 5.1, the slip between the masonry wall and the base block has

been excluded by subtracting LVDT 8 readings from LVDT 1 readings. LVDT 8 readings are illustrated in Fig. 5.2.

As indicated by the hysteresis curves, wall 1 exhibited a relatively ductile behaviour along with a moderate load degradation with respect to the displacement increment. The wall's first yield of the extreme vertical steel at the wall end was monitored and found to be equal to 153 kN and -154 kN in the reverse direction. The first major diagonal crack was observed at a lateral load of 172 kN and at a lateral displacement of about 4 mm. The crack extended at an approximate 45° angle. The diagonal crack in the other direction occurred at -166 kN and at an approximate lateral displacement of -4 mm. The negative sign indicates the reverse direction. Toe crushing occurred at ultimate loads of 200 kN and -209 kN at approximate lateral displacements of 8 mm and -10 mm, respectively. Severe spalling was observed at the wall toes and the base slip for the final loading cycle was small and equal to a maximum value of 10% of the total deformation. Load degradation occurred gradually after the ultimate strengths had been reached and a ductile behaviour was clearly observed. The cracking pattern at the ultimate failure stage can be seen in Fig. 5.3.

5.3 Wall 2

Wall 2 with an aspect ratio of 0.636 and vertical and horizontal reinforcement ratios of 0.4% and 0.2% respectively, was subjected to the displacement history presented in Fig. 4.25 while a constant stress of 0.69 MPa (100 psi) was applied vertically. Fig. 5.4 illustrates the load-top displacement hysteresis curves for wall 2. The lateral displacement was monitored by LVDT 1 while the base slip was monitored by LVDT's 6, 7 and 8. In the relationships illustrated in Fig. 5.4, the slip between the masonry wall and the base block has been excluded; again this was carried out by subtracting LVDT 8 readings from LVDT 1 readings. LVDT 8 readings are illustrated in Fig. 5.5.

As indicated by the hysteresis curves, wall 2 exhibited a relatively brittle behaviour along with considerable load degradation with respect to the displacement

increment. The wall's first yield of the extreme vertical steel was monitored and found to be equal to 251 kN and -274 kN in the reverse direction. The first major diagonal crack was observed at a lateral load of 253 kN and at a lateral displacement of about 4 mm. The crack extended at an approximate 36° angle. The diagonal crack in the other direction occurred at -286 kN and again at an approximate lateral displacement of -4 mm. Toe splitting occurred at ultimate loads of 370 kN and -408 kN at approximate lateral displacements of 15 mm and -10 mm, respectively. Severe spalling was observed at the wall toes and the base slip for the final loading cycle was small and equal to a maximum value of 12% of the total deformation. Load degradation occurred gradually after the ultimate strengths had been reached. The cracking pattern at the ultimate failure stage can be seen in Fig. 5.6.

5.4 Wall 3

Wall 3 with an aspect ratio of 0.467 and vertical and horizontal reinforcement ratios of 0.4% and 0.2% respectively, was subjected to the displacement history presented in Fig. 4.25 while a constant stress of 0.69 MPa (100 psi) was applied vertically. Fig. 5.7 illustrates the load-top displacement hysteresis curves for wall 3. The lateral displacement was monitored by LVDT 1 while the base slip was monitored by LVDT's 6, 7 and 8. In the relationships illustrated in Fig. 5.7, the slip between the masonry wall and the block base has been excluded by subtracting LVDT 8 readings from LVDT 1 readings. LVDT 8 readings are illustrated in Fig. 5.8.

As indicated by the hysteresis curves, wall 3 exhibited a very brittle behaviour along with severe load degradation with respect to the displacement increment. The wall's first yield of the extreme vertical steel was monitored and found to be equal to 474 kN and -526 kN in the reverse direction. The first major diagonal crack was observed at a lateral load of 482 kN and at an approximate lateral displacement of 4 mm. The crack extended at an approximate 45° angle. The diagonal crack in the other direction occurred at -518 kN and again at a lateral displacement of

about -4 mm. Toe splitting occurred at ultimate loads of 553 kN and -529 kN at approximate lateral displacement of 10 mm and -6 mm, respectively. Severe spalling was observed at the wall toes and the base slip for the final loading cycle was about 20% of the total deformation. Load degradation occurred immediately after the ultimate strengths had been reached. The cracking pattern at the ultimate failure stage can be seen in Fig. 5.9.

5.5 Wall 4

Wall 4 had the same aspect ratio of 0.636 as wall 2 but vertical and horizontal reinforcement ratios of 0.6% and 0.2% respectively. It was subjected to the displacement history presented in Fig. 4.25 while a constant stress of 0.69 MPa (100 psi) was applied vertically. Fig. 5.10 shows the load-top displacement hysteresis curves for wall 4. The lateral displacement was monitored by LVDT 1 while the base slip was monitored by LVDT's 6, 7 and 8. In the relationships illustrated in Fig. 5.10, the slip between the masonry wall and the base block has been excluded by subtracting LVDT 8 readings from LVDT 1 readings. LVDT 8 readings are illustrated in Fig. 5.11.

As indicated by the hysteresis curves, the specimen exhibited brittle behaviour along with considerable load degradation with respect to the displacement increment. The wall's first yield of the extreme vertical steel was monitored and found to be equal to 362 kN and -354 kN in the reverse direction. The first major diagonal crack was observed at a lateral load of 379 kN and lateral displacement of 4 mm. The crack extended at an approximate 42° angle. The diagonal crack in the other direction occurred at -331 kN and again at an approximate lateral displacement of -4 mm. Toe splitting occurred at ultimate loads of 415 kN and -409 kN at approximate lateral displacements of 6 mm and -8 mm, respectively. Severe spalling was observed at the wall toes and the base slip for the final loading cycle was small and equal to a maximum value of about 4% of the total deformation. Load degradation occurred shortly after the ultimate strengths had been reached. The cracking pattern at the ultimate failure stage can be seen in Fig. 5.12.

5.6 Wall 5

Wall 5 with an aspect ratio of 0.636 and vertical and horizontal reinforcement ratios of 0.4% and 0.2%, was identical to wall 4 except that it was subjected to a constant stress of 1.73 MPa (250 psi) applied vertically. Fig. 5.13 illustrates the load-top displacement hysteresis curves for wall 5. The lateral displacement was monitored by LVDT 1 while the base slip was monitored by LVDT's 6, 7 and 8. In the relationships illustrated in Fig. 5.13, the slip between the masonry wall and the concrete base was excluded. This was carried out by subtracting LVDT 8 readings from LVDT 1 readings. LVDT 8 readings are illustrated in Fig. 5.14.

As indicated by the hysteresis curves, wall 5 exhibited brittle behaviour along with considerable load degradation with respect to the displacement increment. The first yield of the extreme vertical steel was monitored and found to be equal to 334 kN and -356 kN in the reverse direction. The first major diagonal crack was observed at a lateral load of 337 kN and lateral displacement of about 4 mm. The crack extended at an approximate 42° angle. The diagonal crack in the other direction occurred at -412 kN and again at an approximate lateral displacement of -4 mm. Toe splitting occurred at ultimate loads of 347 kN and -468 kN at approximate lateral displacement of 6 mm and -8 mm, respectively. The difference between the ultimate shear capacities for both loading directions may be caused by various factors such as the cracking propagation mechanism and crack surface conditions that may vary under typical cyclic loading. Severe spalling was observed at the wall toes and the base slip for the final loading cycle was small and equal to a maximum value of about 4% of the total deformation. Load degradation occurred shortly after the ultimate strengths had been reached. The cracking pattern at the ultimate failure stage can be seen in Fig. 5.15.

5.7 Observed Behaviour and Failure Mechanisms

Two types of distinct inelastic behavioural modes have been observed for the test walls. Of the five walls listed in Table 5.1, wall 1 had its failure governed by mixed flexural/shear behaviour since it exhibited flexural yielding of the vertical

steel and compression crushing of masonry at the wall toes as well as diagonal cracking. On the other hand, wall 2, 3, 4 and 5 failures were governed primarily by diagonal tensile cracking causing these walls to fail in a shear dominated mode. Also, at the final loading stage, these walls indicated signs of diagonal compression failure depicted at the areas of the lower two courses of the loaded walls. While these walls failed in a shear dominated mode, they maintained considerable ductility. Table 5.1 summarizes the critical strengths of the test walls.

The shear failure sequence of walls 2, 3, 4, and 5 experienced can be explained as the result of three mechanisms. In the first mechanism, a set of diagonal cracks were formed. Then, a strut mechanism was formed that may consist of one or more struts depending on the wall geometry. Under cyclic loading conditions, walls started losing integrity and major diagonal cracks started opening and extended diagonally from the top to the bottom of the wall. At this time, the strut mechanism became ineffective and the shear resistance was taken over by masonry aggregate interlock at the diagonal crack surfaces and by horizontal steel exposed to severe tension at the diagonal cracks.

Wall 1 experienced a mixed flexure/shear behaviour. Its failure sequence is somewhat similar to the aforementioned brittle failure except that when the diagonal struts formed, and after several energy stabilization cycles, the wall experienced a considerable yield in the extreme vertical steel. At this stage, flexural action dominated and the wall finally failed by toe splitting with extensive elongation of the yielded steel at the extreme ends of the wall.

Also, results indicated a direct relationship between the shear strength of the test walls and the amount of vertical reinforcement as well as the level of axial load. Also, it was noticed that all the test walls experienced their first major diagonal crack at almost the same lateral displacement.

Table 5.1: Key wall test results

Wall No.	Yield Strength (kN)	Diagonal Cracking Strength (kN)	Ultimate Strength (kN)	Final Failure Mode
1	153, - 154	+172, - 166	+200, - 209	Flexure / Shear
2	251, - 274	+253, - 286	+370, - 408	Shear
3	474, - 526	+482, - 518	+553, - 529	Shear
4	362, - 354	+379, - 331	+415, - 409	Shear
5	334, - 356	+337, - 412	+347, - 468	Shear

Fig. 5.1 Load-top wall displacement history of wall 1 excluding displacement due to base sliding.

Fig. 5.2 Load-base slide history for LVDT 8 of wall 1.

Fig. 5.3 Wall 1 at failure stage.

Fig. 5.4 Load-top wall displacement history of wall 2 excluding displacement due to base sliding.

Fig. 5.5 Load-base slide history for LVDT 8 of wall 2.

Fig. 5.6 Wall 2 at failure stage.

Fig. 5.7 Load-top wall displacement history of wall 3 excluding displacement due to base sliding.

Fig. 5.8 Load-base slide history for LVDT 8 of wall 3.

Fig. 5.9 Wall 3 at failure stage.

Fig. 5.10 Load-top wall displacement history of wall 4 excluding displacement due to base sliding.

Fig. 5.12 Wall 4 at failure stage.

Fig. 5.13 Load-top wall displacement history of wall 5 excluding displacement due to base sliding.

Fig. 5.14 Load-base slide history for LVDT 8 of wall 3.

Fig. 5.15 Wall 5 at failure stage.

Chapter 6

FINITE ELEMENT ANALYSES OF THE TEST WALLS

6.1 General

This chapter deals with the numerical modeling of the five test walls using the finite element model developed and discussed in Chapter 3. The modeled shear walls were subjected to monotonic and cyclic displacement sequences and results were compared with the findings from the experimental program. Also, wall crack patterns were predicted numerically and compared with the experimental observations.

6.2 Finite Element Model

As discussed earlier in Section 3.5.2 in the context of spurious mesh-size sensitivity, strain localization can be treated by either using a limiter in the model to prevent it or by modifying the softening parameters accordingly to maintain the equality of energy dissipation with respect to different meshes. Since adjusting the softening parameters is more complicated in the case of compressive fracture

because of the lack of a uniquely defined compressive fracture behaviour, using the deformation localization size limiter strategy appears more convenient. Following Bazant's recommendation that the proper size must be determined experimentally (Bazant, 1976), the finite element size was chosen equal to the experimentally observed dimension of the compressive fracture zone at the base of the wall panels. This procedure is confirmed by the analytical modeling of shear walls carried out by Lotfi and Shing (Lotfi and Shing, 1990) and Seible *et al.* (Seible *et al.*, 1989).

Three finite element meshes to model the wall panels are shown in Fig. 6.1. The modeled walls were subjected to displacement-controlled sequences by specifying horizontal displacement increments at the top nodes while a constant uniformly distributed axial load was applied at the beginning of the incremental analysis as shown in Fig. 6.1. Both monotonic and cyclic displacement sequences are considered in the numerical analyses. Eight node quadrilaterals with 3×3 Gauss points are used in the current analysis. Key material properties utilized in the numerical analyses are given in Table 6.1.

In all the analyses presented in the following subsections, the smeared crack model developed in Chapter 3 was utilized to simulate the reinforced masonry post-crushing and post-cracking behaviour. The model developed by Bathe and Ramaswamy (Bathe and Ramaswamy, 1979) for the concrete compressive stress-strain relationship was adapted for concrete masonry and employed along with the tension-stiffening model based on the crack model developed by Gopalaratnam and Shah (Gopalaratnam and Shah, 1985). Also, the shear retention model developed by Rots and Blaauwendraad (Rots and Blaauwendraad, 1989) assuming a nonlinear function of the tensile strain was employed to model the residual shear at the crack surface. Local concentrations of stresses and force transmitting mechanisms between masonry and reinforcement were also modeled.

6.3 Monotonic Load Analyses

For wall 1, Fig. 6.2 illustrates a comparison between the experimental load-top displacement hysteresis curves reported in Chapter 5 and the monotonic analytical

prediction using the finite element mesh illustrated in Fig. 6.1(a). It can be seen that the analytical model envelope closely matches the experimental results. Also, the numerical model exhibited ductile behaviour similar to the test wall. The ultimate load predicted by the numerical model was 214 kN as compared to the test wall's 209 kN in the positive and -200 kN in the negative loading directions respectively.

In the comparison study between the analytical and experimental results described below, crack patterns predicted analytically at maximum shear resistances are compared with patterns observed experimentally at maximum lateral loads. By employing the numerical model developed in Chapter 3, masonry crack widths were determined numerically at each Gauss point. For wall 1, Fig. 6.3 illustrates a comparison between the analytically predicted crack pattern at a lateral displacement of 5.0 mm (analysed under monotonic displacement-driven increments) and the experimentally observed crack pattern. The analytical crack pattern was developed by employing the GKS graphics library where a computer program was developed to graphically illustrate the crack patterns for the modeled walls. Each line indicates a crack that propagates in a direction represented by the line orientation where the thickness of each line is a graphic representation of the crack width. Note that both crack patterns are reasonably similar and that the model failed eventually in a ductile flexure-dominated mode similar to that experienced experimentally.

For wall 2, Fig. 6.4 illustrates a comparison between the experimental load-top displacement hysteresis curves reported in Chapter 5 and the monotonic analytical prediction using the finite element mesh illustrated in Fig. 6.1(b). It can be seen that while the analytical model envelope closely matches the experimental results up to 75% of the ultimate load, the load-deflection envelope showed slight discrepancies compared to the experimental results above this limit. Also, the numerical model exhibited brittle behaviour similar to the test wall. The ultimate load predicted by the numerical model was 412 kN as compared to the test wall's

370 kN in the positive and -408 kN in the negative loading directions respectively. For wall 2, Fig. 6.5 illustrates a comparison between the analytically predicted crack pattern at a lateral displacement of 6.5 mm (analysed under monotonic displacement-driven increments) and the experimentally observed crack pattern. Note that both crack patterns are reasonably similar and that the model failed in a brittle shear-dominated mode similar to that experienced experimentally.

For wall 3, Fig. 6.6 illustrates a comparison between the experimental load-top displacement hysteresis curves reported in Chapter 5 and the monotonic analytical prediction using the finite element mesh illustrated in Fig. 6.1(c). It can be seen that while the analytical model envelope closely matches the experimental results up to 75% of the ultimate load, the load-deflection envelope showed slight discrepancies compared to the experimental results above this limit. Also, the numerical model exhibited brittle behaviour as experienced by the test wall. The ultimate load predicted by the numerical model was 575 kN compared with 553 kN in the positive and -529 kN in the negative loading directions respectively. For wall 3, Fig. 6.7 illustrates a comparison between the analytically predicted crack pattern at a lateral displacement of 5.5 mm (analysed under monotonic displacement-driven increments) and the experimentally observed crack pattern. Note that both crack patterns are reasonably similar and that the model failed in a brittle shear dominated mode similar to that experienced experimentally.

For wall 4, Fig. 6.8 illustrates a comparison between the experimental load-top displacement hysteresis curves reported in Chapter 5 and the monotonic analytical prediction using the finite element mesh illustrated in Fig. 6.1(b). It can be seen that while the analytical model envelope closely matches the experimental results up to 75% of the ultimate load, the load-deflection envelope showed slight discrepancies compared to the experimental results above this limit. Also, the numerical model exhibited brittle behaviour similar to the test wall. The ultimate load predicted by the numerical model was 437 kN as compared to the test wall's 415 kN in the positive and -409 kN in the negative loading directions respectively.

For wall 4, Fig. 6.9 illustrates a comparison between the analytically predicted crack pattern at a lateral displacement of 5.0 mm (analysed under monotonic displacement-driven increments) and the experimentally observed crack pattern. Note that both crack patterns are reasonably similar and that the model failed in a brittle shear-dominated mode similar to that experienced experimentally.

For wall 5, Fig. 6.10 illustrates a comparison between the experimental load-top displacement hysteresis curves reported in Chapter 5 and the monotonic analytical prediction using the finite element mesh illustrated in Fig. 6.1(b). It can be seen that while the analytical model envelope closely matches the experimental results up to 75% of the ultimate load, the load-deflection envelope showed slight discrepancies compared to the experimental results above this limit. Also, the numerical model exhibited brittle behaviour similar to the test wall. The ultimate load predicted by the numerical model was 483 kN as compared to the test wall's 347 kN in the positive and -468 kN in the negative loading directions respectively. For wall 5, Fig. 6.11 illustrates a comparison between the analytically predicted crack pattern at a lateral displacement of 4.5 mm (analysed under monotonic displacement-driven increments) and the experimentally observed crack pattern. Note that both crack patterns are reasonably similar and that the model failed in a brittle-shear dominated mode similar to that experienced experimentally.

Table 6.2 summarizes key experimental and theoretical strengths for the test walls. In this table, the experimental and analytical yield strengths are the values corresponding to a predefined strain value of 0.22% at the extreme vertical reinforcing bars at the wall ends. While this may be readily understandable from an analytical point of view, the strain recorded experimentally at yield may vary from the predefined value. This may explain the discrepancies in the experimental and analytical results presented in Table 6.2 for the yield strength values.

6.4 Cyclic Load Analyses

In the numerical analyses, the reversed cyclic displacement pattern applied to each model of the test walls was similar to the one employed in the experimental

program except that the stabilization cycles along with the decaying cycles were eliminated. Figs. 6.12 to 6.16 illustrate comparisons between the experimental load-top displacement hysteresis curves reported in Chapter 5 and the fully reversed cyclic loading analytical predictions using the finite element mesh shown in Fig. 6.1.

In general, the analytical predictions for the displacement hysteresis envelopes are in reasonable agreement with the experimental results. Close agreement in the hysteretic load-deformation response can be observed for the relatively ductile wall 1; larger discrepancies between the analytical model and the experimental results for the cyclic load-deformation history are depicted in Figs. 6.13 to 6.16 for the relatively brittle walls 2, 3, 4 and 5. In general, the numerical prediction of the load-displacement hysteresis loops occurred at a lower displacement range compared with those observed experimentally.

6.5 Discussion of Results

6.5.1 *Monotonic Load Analyses*

The analysis of the flexure dominated wall 1 indicated a good match with the experimental results. On the other hand, the analyses of the shear dominated walls 2 to 5 indicated a slight increase in the walls' lateral resistance over the experimental results and this occurred at smaller displacement values. The analytical model successfully captured the sudden drop in the resistance of the shear walls similar to that experienced experimentally soon after reaching their respective ultimate loads. This sudden drop in strength can be explained as the result of widening of the diagonal crack followed by a substantial reduction in the aggregate-interlock forces and in the residual shear at the crack interface. This reduction is a function of the amount of shear reinforcement (mainly the horizontal steel).

This mechanism, to a great extent, was predicted by the current model. Numerical results, using the current model, indicated no spurious shear strength for the modeled shear dominated walls and the sudden drop due to the excessive opening of the diagonal crack has been properly represented.

6.5.2 Cyclic Load Analyses

Even with a good prediction of the wall lateral resistance at various displacement increments, the cyclic numerical model indicated a stiffer behaviour with a lower degree of degradation as compared to that observed experimentally. The discrepancies between the analytical model and the experimental results for the cyclic load-deformation history can be explained by the assumption in the analytical model that cracks close completely upon unloading, a phenomenon that does not occur for the wide shear cracks typically encountered in walls exhibiting a brittle shear response. Adding to the aforementioned, the displacement pattern imposed on the model was somewhat different from the one employed in the experimental program in that the stabilization and decaying cycles were excluded in the analytical model; this means that the energy dissipation process was excluded in the numerical model.

Table 6.1 Key material properties

Test Wall No.	f'_m (MPa)	A_s , vertical		A_s , horizontal		Additional vertical stress (MPa)
		ρ_s (%)	f_y (MPa)	ρ_s (%)	f_y (MPa)	
1	12.0	0.4	438	0.2	436	0.69
2	12.0	0.4	438	0.2	436	0.69
3	12.0	0.4	438	0.2	436	0.69
4	12.0	0.6	440	0.2	436	0.69
5	12.0	0.4	438	0.2	436	1.73

$\epsilon_s = 0.0025$ $f_{cr} = 0.6$ MPa Strain hardening in steel $\alpha = 0.005$

Table 6.2 Key strengths of test walls.

Wall No.	Yield Strength (kN) at Lateral Displacement, Δ (mm)						Ultimate Strength (kN) at Lateral Displacement, Δ (mm)						Final Failure Mode (Exp. & Theo.)
	Experimental			Theoretical			Experimental			Theoretical			
	+ve (kN)	Δ (mm)	-ve (kN)	Δ (mm)	-ve (kN)	Δ (mm)	+ve (kN)	Δ (mm)	-ve (kN)	Δ (mm)	+ve (kN)	Δ (mm)	
1	153	3.15	- 154	- 3.21	163	3.25	200	8.00	- 209	- 10.00	+214	4.50	Flexure/Shear
2	251	4.85	- 274	- 4.16	310	3.50	370	15.00	- 406	- 10.00	+412	6.50	Shear
3	474	4.25	- 526	- 5.86	493	3.50	553	6.00	- 529	- 6.00	+575	5.50	Shear
4	362	3.61	- 354	- 5.10	411	4.25	415	6.00	- 409	- 8.00	+437	5.00	Shear
5	334	3.44	- 356	- 2.70	467	4.00	347	5.70	- 468	- 8.00	+483	4.50	Shear

Fig. 6.1 Test specimens and finite element discretization.

Fig. 6.2 Comparison of analytical and experimental results for wall 1.

Fig. 6.3 Comparison of the analytical and experimental crack patterns for wall 1 at 4.5 mm lateral displacement.

Fig. 6.4 Comparison of analytical and experimental results for wall 2.

Fig. 6.5 Comparison of the analytical and experimental crack patterns for wall 2 at 6.5 mm lateral displacement.

Fig. 6.6 Comparison of analytical and experimental results for wall 3.

Fig. 6.7 Comparison of the analytical and experimental crack patterns for wall 3 at 5.5 mm lateral displacement.

Fig. 6.8 Comparison of analytical and experimental results for wall 4.

Fig. 6.9 Comparison of the analytical and experimental crack patterns for wall 4 at 5.0 mm lateral displacement.

Fig. 6.10 Comparison of analytical and experimental results for wall 5.

Fig. 6.11 Comparison of the analytical and experimental crack patterns for wall 5 at 4.5 mm lateral displacement.

Fig. 6.12 Comparison of cyclic load-deformation behaviour for wall 1.

Fig. 6.13 Comparison of cyclic load-deformation behaviour for wall 2.

Fig. 6.14 Comparison of cyclic load-deformation behaviour for wall 3.

Fig. 6.15 Comparison of cyclic load-deformation behaviour for wall 4.

Fig. 6.16 Comparison of cyclic load-deformation behaviour for wall 5.

Chapter 7

DESIGN IMPLICATIONS AND CODE EVALUATIONS

7.1 General

In this chapter, results from the five test walls are compared with various Code provisions and with experimentally based models developed for the prediction of the lateral resistance of reinforced masonry shear walls. The influences of the design, geometry and loading conditions on the behaviour of shear walls are addressed. Then, two newly proposed experimentally and theoretically based models to predict the strength of reinforced concrete masonry shear walls are presented. Finally, a study to validate the finite element method as a design tool for reinforced masonry shear walls is also presented.

7.2 State-of-the-Art Review on Shear Strength Prediction

The shear strength of reinforced masonry walls depends on their cracking loads as well as on several other mechanisms such as aggregate-interlock forces, dowel action of the vertical reinforcement, force transmission mechanism across cracks

and the truss action of the flexural and shear reinforcement. Experimental evidence has been reported for concrete members (Park and Paulay 1975) and for reinforced masonry shear walls (Shing et al. 1990). The nature of these mechanisms is complex and no unique effective theoretical model has yet been developed to predict the shear strength of a reinforced masonry shear wall.

The Canadian Standards Association in the newly developed Standard S304.1 presents a semi-empirical formula for calculating the shear strength of reinforced masonry walls. The formula recognizes the contribution of the horizontal reinforcement as well as of the axial precompression load to the shear strength of reinforced masonry shear walls. The in-plane shear resistance of a masonry wall (V_r) is given by

$$V_r = \phi_m(v_m bd + 0.25P)\gamma_g + \phi_s(0.6A_v f_y \frac{d}{s}) \quad (7.1)$$

$$V_r < \phi_m(0.4)\sqrt{f'_m}bd\gamma_g \quad (7.2)$$

where d is the effective depth which need not be taken as less than $0.8 l_w$ for walls with flexural reinforcement distributed along the length, γ_g is a factor to account for partially grouted walls, P is the axial compressive load on the section under consideration, and v_m is the shear strength attributed to the masonry given by

$$v_m = 0.16(2 - \frac{M}{Vd})\sqrt{f'_m} \quad (7.3)$$

where M/Vd need not be taken as more than 1 or less than 0.25, and M and V are the factored moment and shear at the section under consideration.

For walls with a low aspect ratio ($h_w/l_w < 1$), the shear capacity can be taken as greater than that given in Eq. 7.2 with a maximum capacity of

$$V_r < \phi_m(0.4)\sqrt{f'_m}bd\gamma_g[2 - \frac{h_w}{l_w}] \quad (7.4)$$

where h_w/l_w should not be taken as less than 0.5 and h_w and l_w are the height and length of the wall, respectively.

The Uniform Building Code (ICBO, 1991) adopted a semi-empirical formula for calculating the shear strength of reinforced masonry walls. The formula recognizes the contribution of the horizontal reinforcement to the shear strength of the wall where the nominal shear strength (V_n) at ultimate is calculated from the following relation:

$$V_n = V_m + V_s \quad (7.5)$$

in which

V_m = the residual strength of masonry after diagonal cracking

V_s = the resistance of the horizontal reinforcement

and

$$V_m = C_d A_n \sqrt{f'_m} \quad (7.6)$$

$$V_s = A \rho_h f_{yh} \quad (7.7)$$

in which A_n is the net horizontal cross-sectional area, ρ_h is the ratio of the horizontal steel with respect to the vertical section through the wall, A is the net or gross horizontal area of the wall based on how the ρ_h value was determined. The coefficient C_d is taken either as $0.2 \sqrt{MPa}$ ($2.4 \sqrt{psi}$) for shear-span-to-depth ratios (M/Vd) less than or equal to 1/4, or as $0.1 \sqrt{MPa}$ ($1.2 \sqrt{psi}$) for (M/Vd) greater than or equal to 1.

Proposed changes to the 1994 National Earthquake Hazard Reduction Program Provisions (*NEHRP*, 1994) include a modified version of the shear strength formula for reinforced masonry walls. The nominal shear strength is given by

$$V_n = V_m + 0.5V_s \quad (7.8)$$

in which the nominal shear strength of masonry (V_m) is given by

$$V_m = c_3(4 - 1.75 \frac{M}{V_d}) A_n \sqrt{f'_m} + 0.25 A_n \sigma_c \quad (7.9)$$

and the shear strength of horizontal reinforcement (V_s) is given by

$$V_s = A \rho_h f_{yh} \quad (7.10)$$

The constant c_3 is equal to $0.083 \sqrt{MPa}$ ($1.0 \sqrt{psi}$) and σ_c is the axial compression stress calculated using the net area A_n . Note that the formula for nominal strength of shear reinforcement (Eq. 7.10) is identical to that in the UBC (Eq. 7.7) except that only 50% of the reinforcement shear strength is assumed to participate effectively in the overall ultimate shear resistance.

As part of the U.S component of the TCCMAR program, Shing et al. (1990) proposed a formula for estimating the in-plane shear capacity of fully-grouted reinforced masonry shear walls based on experimental their study. Their approach recognizes the contributions of the vertical reinforcement as well as of the axial precompression load to the shear strength of reinforced masonry shear walls. The nominal shear strength is given by

$$V_n = V_m + V_s \quad (7.11)$$

in which the nominal shear strength of masonry (V_m) is given by

$$V_m = [c_1(\rho_v f_{yv} + \sigma_c) + c_2] A_n \sqrt{f'_m} \quad (7.12)$$

and the shear strength of horizontal reinforcement (V_s) is given by

$$V_s = [(\frac{l - 2d'}{s}) - 1] A_h f_{yh} \quad (7.13)$$

where ρ_v is the ratio of vertical reinforcement having a nominal yield stress f_{yv} , A_n is the net horizontal area, σ_c is the axial compressive stress calculated using

the net area A_n , l is the length of the wall, d' is the distance of the vertical tension reinforcement to the nearest jamb, s is the spacing of horizontal reinforcement, A_h is the area of a horizontal reinforcing bar having a nominal yield stress f_{yh} , and A_n is the net horizontal area. The empirical constants c_1 and c_2 are equal to $0.0217 \sqrt{MPa}$ ($0.0018 \sqrt{psi}$) and $0.166 \sqrt{MPa}$ ($2.0 \sqrt{psi}$), respectively.

As part of the National Institute of Standards and Technology (NIST) masonry research program, Fattal (Fattal, 1993) developed an empirical expression for predicting the shear strength of fully-grouted and partially grouted masonry shear walls that was based on experimental evidence. The expression is a modification of one developed as part of the Japanese TCCMAR effort (Mutsumura 1985, 1987, 1988) and Fattal demonstrates improvements in accuracy of shear strength predictions with these modifications. The nominal shear stress at ultimate, according to Fattal, is given by

$$v_n = v_m + v_s + v_a \quad (7.14)$$

where v_m , v_s and v_a are the contributions of masonry, horizontal reinforcement and axial compressive load to shear strength, respectively. These values are given by

$$v_m = k_o k_u \left[\left(\frac{0.5}{r + 0.8} \right) + 0.18 \right] \sqrt{f'_m f_{yv}} (\rho_v)^{0.7} \quad (7.15)$$

$$v_s = 0.011 k_o \gamma_g \delta f_{yh} (\rho_h)^{0.31} \quad (7.16)$$

$$v_a = 0.012 k_o f'_m + 0.2 \sigma_c \quad (7.17)$$

where ρ_v and ρ_h are the ratios of vertical and horizontal reinforcement having nominal yield stresses f_{yv} and f_{yh} , respectively. The dimensionless factors k_o , k_u and γ_g depend on the masonry type (i.e clay brick or concrete masonry) and on the percentage of grouting that a wall may have. The factor δ is equal to 1.0 for walls

with points of inflection at mid-height and 0.6 for cantilever walls. The product of v_n and the area A of a horizontal section of the wall based on gross dimensions results in the nominal in-plane shear wall strength V_n and all the parameters f'_m , ρ_v , ρ_h and σ_c must be defined in terms of the gross area of masonry (Mutsumura 1987, 1988).

7.3 Flexural Strength

The flexural strength of a reinforced masonry shear wall can be calculated using simple flexural theory assuming plane sections remain plane after bending. Therefore, the UBC (1988) allows employing the well known rectangular stress block as an approximation of the distribution of compressive stress at the ultimate moment. This block is having a stress level of $0.85 f'_m$ where f'_m is the compressive strength of masonry, and a length of $0.85 c$, where c is the distance of the neutral axis from the extreme compressive fiber and that fibre has a maximum strain of 0.003 according to the UBC. It should be mentioned here that the aforementioned assumptions are similar to the ACI requirements (ACI 318-83) for reinforced concrete.

Based on these assumptions and from the equilibrium conditions, the nominal axial strength P_n and the moment capacity M_n can be calculating as

$$P_n = \gamma \beta f'_m c b - \sum A_{s_i} f_{s_i} \quad (7.18a)$$

$$M_n = \gamma \beta f'_m c b \left(\frac{l}{2} - \frac{\beta c}{2} \right) + \sum A_{s_i} f_{s_i} \left(d_i - \frac{l}{2} \right) \quad (7.18b)$$

where

$$f_{s_i} = E_s \epsilon_u \left(\frac{d_i - c}{c} \right) \text{ for } f_{s_i} \leq f_y \quad (7.19)$$

$$f_{s_i} = E_s \left[\alpha \epsilon_u \left(\frac{d_i - c}{c} \right) + (1 - \alpha) \epsilon_y \right] \text{ for } f_{s_i} > f_y \quad (7.20)$$

in which A_{s_i} is the cross-sectional area of a vertical bar i , d_i is the distance of a bar

from the compression edge, n is the number of reinforcing bars, E_s is the elastic modulus of steel, ϵ_y and f_y are the yield strain and stress of the vertical steel, α is the strain-hardening parameter for steel, γ and β are the stress block parameters, ϵ_u is the maximum allowable strain of masonry at the extreme fiber, and l and b are the length and width of the horizontal cross-section of the wall. In the current study, the maximum strain of masonry is taken as 0.003 as provided in the UBC, the strain-hardening effect is neglected ($\alpha = 0.0$) and the stress block parameters are taken equal to 0.85.

The accuracy of this formula depends on the validity of the stress block as well as on the kinematic assumptions. From finite element analyses, Klammerus (Klammerus, 1988) observed that the variation of the flexural strain with respect to the distance from the neutral axis was far from linear after flexural cracks developed. Shing et al. (Shing et al., 1990) studied the sensitivity of the theoretical results to the stress block parameters and to the linearity of the flexural strain distribution. They concluded that the influence of the parameter β and the distortion in the strain distribution on the interaction curve is negligible, and that the value of γ is significant only when a wall is in the compression dominated regime of the interaction curve. Since shear walls are usually in the tension-dominated regime of interaction curves, the simple flexural formula should be valid and hence was employed in the current study.

Based on Eqs. 7.18 to 7.20, Fig. 7.1 illustrates the axial load-moment capacity interaction curves for the test walls. The comparison of the analytical and experimental results is also illustrated. From this figure it can be observed that the actual failure modes of the specimens are consistent with those given by the interaction curves. Experimental results for wall 1 indicated a mixed flexural/shear behaviour. This is confirmed by the interaction curve of Fig. 7.1(a) where the wall ultimate capacity is located almost on the boundary curve between the flexure and shear zones indicating a mixed failure mode. On the other hand, experimental results for Walls 2, 3, 4 and 5 indicated a shear dominated behaviour. This is confirmed

by the interaction curves of Figs. 7.1(h), (c), (d) and (e) where the walls' ultimate capacities are located in the shear dominated zones of the interaction curves.

7.4 Shear Strength

The shear strength of reinforced masonry walls depends on several factors and mechanisms. The nature of these mechanisms is complex and no unique effective theoretical model has yet been developed to predict the shear strength of a reinforced masonry shear wall. Before diagonal cracking, if the cracking strength happens to be larger than the sum of the residual strength of masonry and the shear resistance of the horizontal reinforcement, then the ultimate shear strength may depend on the cracking load. This cracking load is governed by several factors such as the cracking strength of masonry which is generally assumed to be proportional to $\sqrt{f'_m}$, the aspect ratio of the shear wall, and on the level of the applied axial load. After diagonal cracking, the shear resistance of horizontal reinforcement is activated combined with the dowel action of the vertical flexural reinforcement. Also at the diagonal crack surface, the aggregate-interlock forces contribute to the shear resistance depending on the level of the applied axial stress, the maximum aggregate size, and the crack width.

Among the various parameters controlling the residual shear resistance of a reinforced masonry wall, three parameters are considered: the applied axial compressive stress, the dowel action of the vertical reinforcement, and the amount of the horizontal reinforcement. The first two parameters are directly related to the residual strength of masonry after diagonal cracking, V_m . Therefore, the third parameter can be excluded by calculating the shear resistance of horizontal reinforcement, V_s , and subtracted from the total shear strength, V_u , to obtain the residual strength of masonry after diagonal cracking, V_m .

But what precisely is the shear resistance of the horizontal reinforcement? As indicated in Section 7.2, the shear resistance of the horizontal reinforcement can be determined by various formulas. The CSA S304.1 formula (Eq. 7.1) for nominal shear reinforcement is identical to that employed for concrete except that only

60 % of the reinforcement shear strength is assumed to participate effectively in the overall ultimate shear resistance. On the other hand, the reinforcement shear strength can be overestimated by the UBC formula (Eq. 7.7) which assumes that all the horizontal reinforcement will be activated after diagonal cracking occurs. Also, no rationale was provided in the *NEHRP* (Eq. 7.8) in assuming that only 50 % of the reinforcement shear strength will participate in the overall ultimate shear resistance.

Shing et al. (1990), assuming the top and bottom horizontal bars close to the fixed end do not develop tensile resistance, proposed a formula for predicting the reinforcement shear resistance (Eq. 7.13). In this formula, the reinforcement shear resistance is a function of the distance between the extreme vertical reinforcement bars. In other words, the reinforcement shear resistance is directly proportional to the length of the wall assuming a constant vertical spacing of the horizontal reinforcement. While more rationale is involved in deriving Shing's formula, it overestimates the contribution of the horizontal reinforcement to the overall shear resistance of masonry walls with lower aspect ratios.

In an attempt to rationally predict the contribution of the horizontal reinforcement, twenty four reinforced masonry shear walls of different geometry, design and loading conditions were modeled using the finite element model developed in Chapter 3. A summary of geometry, reinforcement ratios, material properties and loading conditions is presented in Table 7.1. Each wall was loaded numerically until failure and the horizontal reinforcement stress distributions at ultimate failure were determined. It was found that the geometry, represented by the wall aspect ratio, was a contributing parameter to the tensile resistance developed by the horizontal reinforcement. In the absence of any measurements on the horizontal reinforcement of the test walls, it is reasonable to propose the following formula for the shear resistance of horizontal steel:

$$V_s = \frac{1}{(2 - r)} \left[\left(\frac{0.8l}{s} \right) - 1 \right] A_h f_{yh} \quad (7.21)$$

where l is the length of the wall, r is the wall aspect ratio (height/length) and should not be taken less than 0.5 (the range of the analytical study), s is the spacing of horizontal reinforcement, A_h is the area of a horizontal reinforcing bar, f_{yh} is the nominal yield stress of this reinforcement. The ratio $1/(2-r)$ was chosen based on a best fit study of the analytical results while the -1 factor was chosen similar to Shing's reasonable assumption that the top and bottom bars close to the fixed ends do not develop tensile resistance.

To examine the new CSA S304 relations (Eq. 7.2 and Eq. 7.4) for the ultimate shear resistance of masonry walls subjected to in-plane shear, the normalized shear strength $\tau_u/\sqrt{f'_m}$ (where τ_u is obtained by dividing the ultimate shear strength V_u in the stronger direction by the net horizontal cross-sectional area A) was compared with the CSA formulas for the normalized shear resistance of masonry shear walls. To obtain more rational conclusions based on the existing experimental findings, it was decided to present the results of all the walls that failed in a brittle shear dominated failure mode in Fig. 7.2. Note that the 8 square walls tested by Shing et al. (Shing et al., 1990) are shown on the left side and the current 4 test walls are shown on the right side of Fig. 7.2. Also included in the same figure are the reduced shear strengths after applying the resistance reduction factor ϕ_m provided by the CSA Standard S304.1.

While the CSA formula for the shear resistance of walls with aspect ratio of one (Eq. 7.2) appears to be adequate, the CSA formula for aspect ratio less than one (Eq. 7.4) provides insufficient shear resistance without taking into consideration the resistance reduction factor ϕ_m . By including the resistance reduction factor ϕ ($\phi_m = 0.55$, CSA S304.1), safety is assured.

The residual shear strength of masonry V_m was obtained by subtracting the shear resistance of the horizontal reinforcement V_s calculated by means of Eq. 7.21 from the experimentally recorded ultimate shear strength V_u . Then, the residual masonry shear strength τ_m was obtained by dividing V_m by the net horizontal area A ; τ_m was normalized by dividing by $\sqrt{f'_m}$. The resulting normalized masonry

shear strength $\tau_m/\sqrt{f'_m}$ is plotted in Fig. 7.3. As shown in this figure, for walls with higher aspect ratios, the UBC and CSA relations predicted conservative values. On the other hand, for walls with lower aspect ratios, both the UBC and CSA relations predicted close residual strengths and were less conservative as compared to those for walls with high aspect ratios.

As discussed earlier, to examine the influence of the applied axial compressive stress and of the amount of vertical steel on the residual strength of masonry, the normalized masonry strength $\tau_m/\sqrt{f'_m}$ is plotted against the axial strength σ_c in Fig. 7.4 and against the parameter $\rho_v f_{yv}$ (where ρ_v is the ratio of the vertical reinforcement and f_{yv} the associated yield stress) in Fig. 7.5. To achieve more accurate and rational conclusions based on the existing experimental findings, it was decided to present the results of all the walls that failed in a brittle shear dominated failure mode in Figs. 7.4 and 7.5. Note that the 8 square walls tested by Shing et al. (Shing et al., 1990) which failed in a brittle shear dominated failure mode are shown on the left and the current 4 test walls are shown on the right of Figs. 7.4 and 7.5.

Employing least-squares fit, it was found that the normalized masonry strength increases with respect to the increase in the axial stress σ_c with an average rate of $0.03 (\sqrt{MPa})(0.0025 \sqrt{psi})$. Also, the normalized masonry strength increases slightly with respect to the increase in the value $\rho_v f_{yv}$ with an average rate of $0.01 (\sqrt{MPa})(0.0008 \sqrt{psi})$. Based on these findings, a new formula for the residual shear strength of masonry V_m is proposed:

$$V_m = [0.01\rho_v f_{yv} + 0.03\sigma_c + v_m]A_n\sqrt{f'_m} \quad (7.22)$$

where

$$v_m = 0.16(2 - h/l) \quad (7.23)$$

In this formula, the effects of fracture strength of masonry, dowel action of

the vertical reinforcement, axial stress level and wall aspect ratio are included in predicting the masonry residual strength.

Employing Eqs. 7.21, 7.22 and 7.23, the nominal shear strengths V_n are evaluated for the five test walls and compared with the experimental results as illustrated in Fig. 7.6(a). It can be seen that close agreement exists between the experimental and calculated ultimate capacities of the test walls. The formulas thus capture the overall variation in shear strength with respect to different geometry, other design variables and axial load levels.

7.5 Evaluation of Various Codes and Models for Shear Wall Strength Predictions

The nominal shear strengths of the five test shear walls were evaluated according to the CSA Standard, UBC, and *NEHRP* recommendations. Results are illustrated in Fig. 7.6(a) along with a comparison with the proposed formula predictions. While the UBC typically underestimates the shear strengths of the test walls and therefore is conservative, the CSA Standard slightly overestimates their strengths and therefore is somewhat less conservative. The *NEHRP* relations resulted in a good match with the experimental results as well as with the proposed formula predictions.

Fig. 7.6(b) illustrates the predictions of the nominal shear strengths of the test walls based on Shing's formula (Shing et al., 1990) and on Fattal's formula (Fattal, 1993). Both Shing's and Fattal's formulas overestimate the shear resistances of the test walls.

Fig. 7.6 illustrates various Code predictions for the ultimate shear strength of masonry shear walls. While these Codes have upper limits for their predictions of the nominal shear strengths, their predictions of the residual masonry shear strength, V_m , and the shear resistance of the horizontal reinforcement, V_s , are not reflected in this figure. Therefore, the residual shear strengths of the test walls, V_m , were calculated using various approaches as depicted in Figs. 7.7(a) and (b).

As illustrated in Figs. 7.7(a) and (b), the UBC provides conservative values for

the residual shear strength of masonry while the CSA and NEHRP values are close to those predicted by the new proposed formula. Also, while Shing's model predicts well the residual shear strength of masonry walls with an aspect ratio equal to one (Shing 1990), it is too conservative for shear walls having lower aspect ratios. Also, while Fattal's model predicts well the residual shear strength of masonry for shear walls 1, 2, and 3, the model overestimates the influences of vertical reinforcement and axial stress level on the residual shear strength of masonry as shown in Fig. 7.7(b) for walls 4 and 5.

Similarly, Fig. 7.8 illustrates various Code and model predictions of the shear resistance of horizontal reinforcement, V_s . While the UBC appears to overestimate the contribution of the horizontal reinforcement to the shear strength of masonry shear walls, the CSA and NEHRP relations provide reasonably accurate predictions. Again, while the model proposed by Shing predicts well the horizontal reinforcement shear resistance for walls with an aspect ratio equal to one, it overestimates the contribution of the horizontal reinforcement for walls with lower aspect ratios as shown in Fig. 7.8(b). Fattal's work on the other hand provides better agreement.

From the comparison of results illustrated in Figs. 7.6, 7.7 and 7.8, it can be concluded that existing code and model formulas for the ultimate shear resistance of reinforced masonry shear walls do not provide an accurate measure for either the determination of the residual masonry strength or for the estimation of the contribution of the horizontal reinforcement to the shear resistance.

7.6 Finite Element Method as a Design Tool

As discussed in Chapter 6, the numerical modeling of the five test walls indicated an acceptable prediction of their ultimate shear strengths. Fig. 7.9 illustrates a comparison between the FEM predictions and the experimental results as well as the shear strength values predicted employing the proposed experimentally based formula for the shear strength of reinforced masonry walls. As shown in this figure, the FEM results provide a reasonably good match to the experimental results; they

slightly overestimate the shear strength with a maximum error between 1.5 and 5%.

Using the FEM approach of this thesis as a tool to further investigate the influence of various design parameters on the shear strength of reinforced masonry shear walls, 24 wall cases were analyzed for an extended range of these parameters: wall aspect ratio, percentage of horizontal reinforcement, percentage of vertical reinforcement, and level of the applied axial vertical load. A summary of the modeled cases is given in Table 7.1.

To examine the influence of the applied axial compressive stress and amount of vertical steel on the residual strength of masonry, the normalized masonry strength $\tau_m/\sqrt{f'_m}$ is plotted against the axial strength σ_c in Fig. 7.10 and against the parameter $\rho_v f_{yv}$ in Fig. 7.11 where ρ_v is the ratio of the vertical reinforcement and f_{yv} is the yield stress.

The residual masonry shear strength τ_m was obtained by subtracting the shear resistance of the horizontal reinforcement, V_s , calculated by Eq. 7.21 from the FEM predicted ultimate shear strength, V_n , to obtain the value of the residual masonry strength, V_m . Then, V_m was divided by the net horizontal area A_n and was normalized by $\sqrt{f'_m}$.

Employing least-squares fit, it was found that the normalized masonry strength increases with respect to the increase in the axial stress σ_c with an average rate of $0.04 \sqrt{MPa}$ ($0.0034 \sqrt{psi}$). Also, the normalized masonry strength increases with respect to the increase in the value $\rho_v f_{yv}$ with an average rate of $0.02 \sqrt{MPa}$ ($0.0017 \sqrt{psi}$). Based on the previous findings, a new formula for the residual shear strength of masonry, V_m , is proposed:

$$V_m = [0.02\rho_v f_{yv} + 0.04\sigma_c + v_m]A_n\sqrt{f'_m} \quad (7.24)$$

where v_m can be calculated using Eq. 7.23.

Employing Eqs. 7.21, 7.23 and 7.24, the nominal shear strengths, V_n , are eval-

uated for the five test walls and compared with the experimental results as shown in Fig. 7.12. While the aforementioned formula (Eq. 7.24) slightly overestimates the ultimate shear resistance of the test walls by a maximum error of about 5%, the formula captures the overall variation of the shear strength with respect to different geometry, design variables and axial stress level. By employing a resistance reduction factor (similar to the ϕ_m factor of CSA Standard S304.1), the finite element method could be very useful as a design tool that could avoid costly experimental studies.

Employing Eqs. 7.17 to 7.20, Fig. 7.13 illustrates the axial load-moment capacity interaction curves for the modeled 24 walls grouped in to six interaction curves representing three aspect ratios and two vertical reinforcement ratios. As illustrated in Fig. 7.13, the failure mechanism of a reinforced masonry wall can be changed from a flexural mode to a shear mode by increasing the axial load or by reducing the amount of the horizontal reinforcement. Therefore, to provide adequate safety for walls subject to brittle shear behaviour, the highest axial load that could possibly be induced in a wall should be taken into consideration during design.

Table 7.1 Summary table of modeled wall properties

Wall No.	Aspect Ratio	A _s , vertical		A _s , horizontal		Vertical Stress (MPa)
		ρ_h (%)	f_y (MPa)	ρ_v (%)	f_y (MPa)	
1	1.0	0.1	438	0.4	436	0.69
2	1.0	0.1	438	0.4	436	1.73
3	1.0	0.1	440	0.6	436	0.69
4	1.0	0.1	440	0.6	436	1.73
5	1.0	0.2	438	0.4	436	0.69
6	1.0	0.2	438	0.4	436	1.73
7	1.0	0.2	440	0.6	436	0.69
8	1.0	0.2	440	0.6	436	1.73
9	0.636	0.1	438	0.4	436	0.69
10	0.636	0.1	438	0.4	436	1.73
11	0.636	0.1	440	0.6	436	0.69
12	0.636	0.1	440	0.6	436	1.73
13	0.636	0.2	438	0.4	436	0.69
14	0.636	0.2	438	0.4	436	1.73
15	0.636	0.2	440	0.6	436	0.69
16	0.636	0.2	440	0.6	436	1.73
17	0.467	0.1	438	0.4	436	0.69
18	0.467	0.1	438	0.4	436	1.73
19	0.467	0.1	440	0.6	436	0.69
20	0.467	0.1	440	0.6	436	1.73
21	0.467	0.2	438	0.4	436	0.69
22	0.467	0.2	438	0.4	436	1.73
23	0.467	0.2	440	0.6	436	0.69
24	0.467	0.2	440	0.6	436	1.73

$$f'_m = 12.0 \text{ MPa}$$

$$\epsilon_o = 0.0025$$

$$f_{cr} = 0.6 \text{ MPa}$$

$$\text{Strain hardening in steel } \alpha = 0.005$$

(a) Wall No. 1

(b) Wall No. 2

(c) Wall No. 3

(d) Wall No. 4

(e) Wall No. 5

Fig. 7.1 Axial load-moment capacity interaction curves for the test walls.

Fig. 7.2 Ultimate strength of masonry.

Fig. 7.3 Residual strength of masonry.

Fig. 7.4 Influence of axial stress on the residual strength of masonry.

Fig. 7.5 Influence of vertical steel on the residual strength of masonry.

(a)

(b)

Fig. 7.6 Different code and model predictions for ultimate shear strength of masonry shear walls.

(a)

(b)

Fig. 7.7 Different code and model predictions for residual shear strength of masonry.

(a)

(b)

Fig. 7.8 Different code and model predictions for horizontal reinforcement shear resistance.

Fig. 7.9 Finite element method predictions for ultimate shear strength of test walls.

Fig. 7.10 Predicted FEM influence of axial stress on the residual strength of masonry.

Fig. 7.11 Predicted FEM influence of vertical steel on the residual strength of masonry.

Fig. 7.12 FEM based formula for prediction of ultimate shear strength of masonry shear walls.

(a)

(b)

(c)

(d)

(e)

(f)

Fig. 7.13 Numerical study of the effect of axial load and horizontal reinforcement ratio on lateral load resistance.

Chapter 8

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 General

The design of reinforced masonry shear walls for all critical seismic limit states requires a detailed understanding of the complete reversed cyclic response. Therefore, large scale experimental data are required to establish behavioural characteristics of masonry shear walls subjected to cyclic loading before comprehensive and representative design models can be developed. Since cost and laboratory facilities impose significant limitations on the extent of full-scale testing, current research efforts are mainly concentrated on studying experimentally the physical behaviour of masonry components and utilizing the experimental results to verify and validate up-to-date simple or complex analytical models. Then these models can be employed to perform a large-scale three-dimensional computer simulation utilizing state-of-the-art computing facilities to predict the behaviour of reinforced masonry

structures under monotonic and cyclic/seismic loading patterns up to the ultimate stage.

The aforementioned problem is highly nonlinear in nature and various models have been developed to trace this behaviour. The finite element method offers a powerful tool for modeling complex structural systems under various loading patterns and tracing the complete nonlinear behaviour up to the ultimate stage. Using this technique, various physical characteristics of reinforced masonry such as post-cracking or post-crushing phenomena can successfully be modeled.

A review of the available literature showed that little work has been carried out to simulate the behaviour of reinforced masonry shear walls undergoing progressive failure under load reversals. Also, very few reliable experimental results indicating the influence of various parameters on the behaviour of masonry shear walls under cyclic loading are available. The current work was undertaken with the objectives to develop a rational analytical model to accurately trace the behaviour of reinforced masonry shear walls, to experimentally investigate the influence of key parameters affecting wall behaviour, and to assess the developed model against the experimental findings. A summary of the major phases of this research work and the conclusions drawn are given in the following section.

8.2 Summary and Conclusions

A smeared crack model approach has been used in finite element analyses of concrete structures. While such a technique proved to be reliable and numerically efficient in many instances, its capability in predicting the mechanical behaviour for a variety of matrix-aggregate composites has not been thoroughly studied especially in the field of complex composite materials such as reinforced masonry.

While numerous problems are associated with the smeared crack model such as mesh-size dependency of numerical solution, failure to model elements with dominant shear failure and spurious kinematic modes, it is still the simplest and most efficient approach to model the fracture behavior of reinforced masonry shear

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PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
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walls exhibiting diffuse crack patterns.

In the current study, a smeared crack model for reinforced masonry was developed. Most of the newest developments in the field of reinforced masonry modeling, such as fixed, coaxial rotating, non-coaxial rotating crack representation were incorporated. The Modified Compression Field Theory (MCFT) was implemented. Numerous constitutive models for reinforced masonry under tension and compression were incorporated into the program and their significance on the numerical solution was assessed. Post-tension cracking and post-compression crushing behaviour have been modeled. Strain-softening behaviour and different tension-stiffening models were incorporated in the numerical solution. Aggregate interlock and force transmitting mechanisms at crack surface were modeled. Also, the influence of the anisotropy of partially grouted masonry was considered. The capabilities of the model in capturing various failure mechanisms of reinforced masonry shear walls undergoing different monotonic and cyclic loading patterns were evaluated.

The accuracy of this model was verified against available experimental evidence as well as against data from the concurrent experimental program. This experimental program involved testing five reinforced masonry shear walls that were fully grouted. The program was carried out to experimentally investigate the influence of the wall aspect ratio (wall height/length ratio), the amount of vertical reinforcement and the level of axial compressive stress on the performance of reinforced masonry shear walls subjected to cyclic loading while undergoing static vertical loading. These results along with the available published experimental data provided the basis to verify the numerical model developed as a first phase of the thesis.

The analytical predictions were compared with the response of the tested walls. The design implications of the experimental findings were discussed and rational mathematical models to describe the behaviour of reinforced masonry shear walls subjected to lateral in-plane cyclic loading were developed.

Based on the experimental findings and the analytical study of the behaviour

of reinforced masonry shear walls, the following conclusions are reached:

1. For flexure dominated walls, the analytical load-displacement history matches well the experimental envelope curve. The vertical steel yielding pattern due to the widening of flexural cracks and the extent of the crushing is very similar to that observed experimentally. Close prediction for the wall maximum load resistance was found.
2. For shear dominated walls, the analytical load-displacement history matches well the experimental envelope curve. The maximum lateral load resistance predicted by the analytical model is slightly higher than that shown in tests and occurs at a lower lateral displacement value. Numerical results, using the current model, indicated no spurious shear strength for the modeled shear dominated walls and the sudden load decrease due to the excessive opening of the diagonal crack has been properly captured.
3. Even with a good prediction of the wall lateral resistance at various displacement increments, the cyclic numerical model indicated a stiffer behaviour with a lower degree of degradation as compared to that observed experimentally. The discrepancies between the analytical model and the experimental results for the cyclic load-deformation history can be explained by the assumption in the analytical model that cracks close completely upon unloading, a phenomenon that does not occur for wide shear cracks typically encountered in walls exhibiting a brittle shear response.
4. A detailed parametric study was conducted to calibrate the analytical model and to demonstrate phenomenological performance characteristics in conjunction with empirical and experimental data. Based on this study, the following conclusions can be stated:
 - The study of the effect of three different tension-softening models on the load-displacement history indicated that they have little influence on the global behaviour of a shear-dominated wall panel and that they focus mainly on the pre-peak region.

- On the other hand, the influence of the shear retention factor on the shear-dominated wall panel has focused only on the post-peak region where it affects the rate of degradation of the wall lateral load resistance. The non-coaxial shear retention model that is based on a nonlinear degradation function showed a better performance over other models.
 - The study of the influence of various compressive stress-strain relationships of masonry on the load-displacement history of shear walls indicated that the compressive constitutive law has a slight effect on the load-deformational characteristics of shear walls failing in flexure. For walls failing in a brittle shear fashion, only Bathe's model indicated a good match with the experimental results especially in the region of the load displacement history above 70% of the peak load. This was due to the fact that the aforementioned model provides a smaller value for the tangential stiffness at a given strain compared to other models and this in turn made the analytical model more flexible.
5. From the experimental study that involved testing five masonry walls, the following conclusions are reached:
- A linear relationship can be assumed between wall aspect ratio and lateral load resistances for walls having the same reinforcement ratios and subjected to the same vertical stress level.
 - While the walls of the test group with aspect ratios of less than 1.0 failed in a shear dominated mode, they maintained considerable ductility.
 - Even with increased length and increased axial vertical stress, the walls experienced their first major diagonal cracks at almost the same lateral displacements.
 - While increasing the vertical reinforcement ratio has a slight effect on the wall lateral load resistance, increasing the amount of vertical load increases the wall lateral load resistance considerably.
6. Based on an evaluation of various Code relations and model predictions of the shear resistance of the five test walls combined with results from sixteen walls tested at the University Colorado, the following conclusions are reached:

- While the UBC underestimated the shear strength of the test walls, the CSA Standard overestimated their shear strengths. The *NEHRP* predictions provide a good match with the experimental results and tend to slightly overestimate the shear strengths. Shing's and Fattal's formulas overestimated the shear resistance of the walls.
 - It was found that different code and model predictions for the ultimate shear resistance of reinforced masonry shear walls do not provide an accurate measure of either the residual masonry strength or the contribution of the horizontal reinforcement to the shear resistance.
7. Based on the experimental findings from the five test walls as well as the sixteen University of Colorado walls, a new formula was proposed for the residual shear strength of masonry shear walls. The proposed formula appears to be more reliable and consistent with the experimental results and recognizes the contribution of the vertical reinforcement, the axial load and the wall aspect ratio on the resistance of masonry shear walls.
 8. By employing the numerical model, an analytical parametric study was conducted to investigate the influence of four parameters, namely wall aspect ratio, amount of vertical reinforcement, horizontal reinforcement ratio and axial vertical stress level, on the residual shear strength of masonry shear walls. An analytical finite element based theoretical formula was proposed for the residual shear strength of reinforced masonry shear walls. Results from the proposed formula were compared with the experimental findings from the five test walls. There was close agreement between the FEM and the experimental results with only a slight overestimation of wall shear strength.

8.3 Recommendations for Future Work

The following experimental and analytical studies are recommended:

1. Determine experimentally the contribution of the horizontal reinforcement ratio to the shear strength of masonry shear walls for walls with low aspect ratios.

2. Determine experimentally the effect of relative compressive strength of grout versus masonry strength on the overall behaviour of reinforced masonry shear walls.
3. Study experimentally the behaviour of partially grouted masonry walls and compare with the behaviour of fully grouted walls.
4. Test walls with low aspect ratios and higher levels of axial compressive stress since only two vertical load levels were employed in the course of this study.
5. Test walls with low aspect ratios and higher amounts of vertical reinforcement since only two vertical reinforcement ratios were employed in the course of this study.
6. Refine the analytical cyclic reversal model beyond the assumption that cracks close completely upon unloading, a phenomenon that does not occur for wide shear cracks typically encountered in walls with brittle shear response.
7. Finally, determine experimentally failure envelopes for masonry under biaxial loading conditions.

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