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Seismic Analysis and Design of Post-tensioned Concrete Masonry Walls

By Peter T. Laursen

A thesis submitted in partial fulfilment of the requirements for the Degree of Doctor of Philosophy

Supervised by Dr. Jason M. Ingham

University of Auckland
Department of Civil and Environmental Engineering
New Zealand
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ABSTRACT

This thesis explores the seismic analysis and design of post-tensioning concrete masonry (PCM) walls. Using unbonded post-tensioning, walls are vertically prestressed by means of strands or bars which are passed through vertical ducts inside the walls. As the walls are subjected to lateral displacements (in-plane loading), gaps form at the horizontal joints, reducing the system stiffness. As long as the prestressing strands are kept within the elastic limit, or at least maintain a considerable amount of the initial prestressing force, they can provide a restoring force, which will return the walls to their original alignment upon unloading. The key feature in this behaviour is attributable to the tendons being unbonded over the entire wall height, allowing for distribution of tendon strain over the entire length of the tendon.

An extensive literature review found that post-tensioning of masonry has had limited application in seismic areas and that there currently are no specific code requirements for it's use for ductile seismic design, largely as a consequence of little knowledge about the ductility capacity and energy dissipation characteristics. It was concluded that structural testing of PCM walls and concrete masonry creep and shrinkage testing were essential to advance the understanding of this construction type.

Creep and shrinkage experiments confirmed that long term prestress losses are considerable in both grouted and ungrouted concrete masonry, and must be taken into account in design. It was concluded that it is essential to use high strength steel for prestressing of PCM in order to reduce long term losses.

Structural testing confirmed that fully grouted unbonded post-tensioned concrete masonry is a competent material combination for ductile structural wall systems. In particular, PCM walls strengthened in the flexural compression zones with confining plates are expected to successfully withstand severe ground shaking from an earthquake. It was suggested that partially and ungrouted PCM walls may suitably be used in strength design (non-ductile).

The proposed prediction method for wall in-plane behaviour was validated by experimental results. Good correlation between predictions and results was found. Displacement spectra were developed for ductile seismic design of PCM walls. These can be used to accurately estimate the displacement demand imposed on multi-storey PCM cantilever walls.
DISCLAIMER

This thesis was prepared for the Department of Civil and Environmental Engineering at the University of Auckland, New Zealand, and describes analysis and design of post-tensioned concrete masonry walls. The opinions and conclusions presented herein are those of the author and do not necessarily reflect those of the University of Auckland or any of the sponsoring parties to this project.

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ADVISORY COMMITTEE

Dr. J.M. Ingham, Senior Lecturer (principal advisor)
Dr. B.J. Davidson, Senior Lecturer
Assistant Professor R. C. Fenwick
Professor B. Melville
Mr. E. Lapish, Consultant Engineer

EXAMINATION COMMITTEE

Assistant Professor C.E. Ventura, Department of Civil Engineering, University of British Columbia, Canada (chair)
Dr. J.M. Ingham, Senior Lecturer, University of Auckland
Dr. H.R. Ganz, VSL (Switzerland) Ltd.
Professor M.J.N. Priestley, University of California, San Diego, USA
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**NOTATION**

Roman characters:

- \( a \) equivalent rectangular stress block compression zone length
- \( a_g \) seismic ground acceleration
- \( A_g \) gross area of wall cross section
- \( a_o \) flexural compression zone length at \( M_o \)
- \( A_p \) confining plate area for Priestley-Elder stress-strain curve
- \( A_{ps} \) total prestressing steel area in wall cross section
- \( A_{psj} \) prestressing tendon area of tendon ‘j’
- \( A_r \) wall aspect ratio
- \( A_s \) shear (horizontal) reinforcing steel area
- \( a_y \) flexural compression zone length at \( M_y \)
- \( b_w \) wall thickness
- \( c \) distance from extreme masonry fibre in compression to flexural neutral axis
- \( C_c \) concrete masonry creep coefficient
- \( c_e \) distance from extreme masonry fibre in compression to flexural neutral axis at \( M_e \)
- \( d \) effective wall length for calculation of \( V_{st} \)
- \( d \) lateral displacement at \( h_e \)
- \( D \) displacement level
- \( d_{cr} \) total lateral displacement at \( h_e \) due to \( V_{cr} \)
- \( d_{crfl} \) lateral flexural displacement at \( h_e \) due to \( V_{cr} \)
- \( d_{crsh} \) lateral shear displacement at \( h_e \) due to \( V_{cr} \)
- \( d_e \) total lateral displacement at \( h_e \) due to \( V_e \)
- \( d_{efl} \) lateral flexural displacement at \( h_e \) due to \( V_e \)
- \( d_{esh} \) lateral shear displacement at \( h_e \) due to \( V_e \)
- \( d_{h*} \) lateral displacement demand at the equivalent SDOF structure height
- \( d_j \) lateral displacement of floor ‘j’
- \( d_n \) total lateral displacement at \( h_e \) due to \( V_f \)
- \( d_{nfl} \) lateral flexural displacement at \( h_e \) due to \( V_f \)
- \( d_{nsh} \) lateral shear displacement at \( h_e \) due to \( V_f \)
- \( d_o \) lateral displacement increment at flexural overstrength
- \( d_r \) roof lateral displacement demand
- \( d_{tg} \) target displacement demand
- \( d_{ty} \) lateral displacement increment at first tendon yield
- \( d_y \) lateral displacement at first tendon yield
d_u \quad \text{ultimate wall displacement capacity}

d_{v_{\text{max}}} \quad \text{measured wall displacement at } V_{\text{max}}

E_I \quad \text{elastic stiffness of wall cross section}

E_m \quad \text{masonry elastic modulus}

\bar{E}_m \quad \text{average masonry elastic modulus related to Priestley-Elder stress-strain curve}

E_{ps} \quad \text{prestressing steel elastic modulus}

e_i \quad \text{total tendon force eccentricity with respect to wall centre line}

e_{ic} \quad \text{shortest horizontal distance between any tendon and compression end of wall}

e_{te} \quad \text{horizontal distance between extreme tendon in tension and compression end of wall}

e_j \quad \text{horizontal distance between tendon ‘j’ and compression end of wall}

f'_{cb} \quad \text{concrete masonry unit crushing strength}

f'_{cg} \quad \text{grout cylinder crushing strength, specific to AS 3700}

f'_g \quad \text{grout cylinder crushing strength}

f'_j \quad \text{mortar cylinder crushing strength}

f'_m \quad \text{masonry crushing strength}

f'_{uc} \quad \text{unconfined masonry unit crushing strength, specific to AS 3700}

f_m \quad \text{axial masonry stress}

f_{mf} \quad \text{final prestress at termination of creep and shrinkage experiment}

f_{mi} \quad \text{masonry stress immediately after prestressing}

F_n \quad \text{generic symbol for ‘function of’}

F_{n_{cr}} \quad \text{time development function for creep}

F_{n_{sh}} \quad \text{time development function for shrinkage}

f_{ps} \quad \text{instantaneous prestressing steel stress}

f_{psi} \quad \text{prestressing steel stress immediately after anchorage lock-off}

f_{pu} \quad \text{ultimate (rupture) strength of prestressing steel}

f_{py} \quad \text{yield strength of prestressing steel}

f_{se} \quad \text{average tendon stress at } V_f \text{ for wall in unloaded state}

f_{vy} \quad \text{nominal yield strength of shear reinforcing steel}

f_y \quad \text{nominal yield strength of reinforcing steel}

f_{yn} \quad \text{nominal yield strength of confining steel}

h^* \quad \text{equivalent height of SDOF structure}

h'' \quad \text{horizontal confined dimension}

h_{cr} \quad \text{location of first cracking height for applied moment of } M_e

h_c \quad \text{wall equivalent height}

h_n^* \quad \text{equivalent height of modal mass of } n^{th} \text{ mode}

h_p \quad \text{vertical extent of plastic deformation zone at wall ultimate displacement}
\(h_s\)  vertical distance between floors n and n-1
\(h_w\)  wall height
k  defining the maximum permissible extreme masonry strain \(k_f^e\) at \(M_e\)
K  prism strength and strain enhancement factor for Priestley-Elder stress-strain curve
\(K_1\)  initial stiffness of bilinear elastic SDOF
\(K_2\)  post-yield stiffness of bilinear elastic SDOF
\(k_c\)  concrete masonry specific creep
\(K_d\)  similar to K but related to high strain rate
\(k_r\)  prestressing steel relaxation parameter
\(l_j\)  length of tendon 'j'
\(l_w\)  wall length
M  base moment
\(M^*\)  applied factored moment
\(M^*\)  equivalent mass of SDOF structure
\(M_{cr}\)  first cracking moment
\(M_e\)  maximum serviceability moment
\(M_{max}\)  maximum developed base moment from time-history analysis
\(M_n\)  nominal strength of wall (moment)
\(M_{n^*}\)  modal mass of nth mode
\(M_o\)  wall base moment overstrength
\(M_t\)  total seismic horizontal mass
\(M_{by}\)  wall base moment increase at first tendon yield
\(M_y\)  wall base moment at first tendon yield
\(M_{y^*}\)  required yield moment strength of nominally elastic SDOF structure
\(M_{y^*}\)  provided yield moment strength for SDOF structure
N  axial load due to wall self-weight, and live and dead load from suspended floors
\(N^*\)  applied factored axial force
\(N_e\)  externally applied axial load in Series 3 experiments
\(N_w\)  wall self-weight in Series 3 experiments
P  prestress force in wall
\(P^*\)  applied factored prestress force
\(P_i\)  prestressing force immediately after anchorage lock-off
\(P_j\)  prestressing force in tendon 'j' (unloaded state)
\(P_l\)  prestressing force after all loss has occurred
\(P_y\)  total tendon force when stress in all tendons is \(f_{py}\)
R
- force reduction factor associated with ductile seismic design

ra
- concrete roughness amplitude of construction joint (shear friction calculation)

s
- vertical spacing of shear reinforcing steel

Sa
- elastic spectral acceleration

Sd
- spectral displacement

Sh
- vertical confined dimension (confining plate spacing)

T
- fundamental structural period

Tl
- first mode natural period

t
- time

t0
- time of application of prestress to masonry

t1
- time at which all time dependent prestress loss has occurred
	n
- time at time step n

ue
- vertical extension of 'tension' end of wall at Vf

uj
- vertical displacement of top anchorage point of tendon 'j' at Vf

us
- vertical shortening of 'compression' end of wall at Vf

V
- applied lateral force at he

V*
- applied factored shear force

Vbase
- base shear due to lateral forces

Vcr
- wall lateral force at he corresponding to Mcr

Ve
- wall lateral force at he corresponding to Me

Vf
- lateral force applied at he corresponding to Mn

vm
- masonry shear strength (stress)

Vm
- wall shear strength (force) due to masonry

vmax
- maximum measured wall shear stress

Vmax
- maximum experimental lateral force

Vmax
- estimated maximum base shear at wall overstrength (shear friction calculation)

Vmax
- maximum developed base shear from time-history analysis

vnehp
- shear strength (stress) predicted by NEHRP provisions

vnzs
- shear strength (stress) predicted by NZS 4230:1990 provisions

Vo
- wall lateral force at he corresponding to Mo

vp
- shear strength (stress) predicted by Paulay and Priestley provisions

Vs
- wall shear strength (force) due to contribution from Vm and Vs

Vst
- wall shear strength (force) due to horizontal reinforcing steel

Vty
- wall lateral force at he at first tendon yield

Vy
- wall lateral force at he corresponding to My
required yield strength of nominally elastic SDOF structure
provided yield strength for SDOF structure
horizontal location of tendon ‘j’ with respect to wall centre line
seismic zone factor (NZS 4203:1992)
slope of descending branch of Priestley-Elder stress-strain curve
similar to \( Z_m \) but related to high strain rate
vector indicating location (height) of masses given by \( \{ m \} \)
mass vector of MDOF structure

Greek characters:
\( \alpha \) defines equivalent rectangular stress block average stress \( \alpha f'_{m} \)
\( \alpha \) strain hardening ratio for bilinear SDOF structure
\( \beta \) defines equivalent rectangular stress block length \( a = \beta c \)
\( \Delta e_{py} \) strain increase in extreme tendon at first tendon yield
\( \Delta f_{cr} \) long term prestress loss due to creep
\( \Delta f_{pl} \) total long term prestress loss
\( \Delta f_{pr} \) long term prestress loss due to prestressing steel relaxation
\( \Delta f_{sh} \) long term prestress loss due to shrinkage
\( \Delta P \) total tendon force increase at \( M_n \)
\( \Delta P_j \) tendon force increase of tendon ‘j’ at \( V_f \)
\( \Delta P_{ty} \) total tendon force increase a first tendon yield
\( \Delta P_{tyj} \) force increase in tendon ‘j’ at first tendon yield
\( \Delta t_n \) length of time step \( n \)
\( \varepsilon \) masonry axial strain
\( \varepsilon_{cr} \) long term concrete masonry creep strain
\( \varepsilon_{m} \) masonry strain at maximum prism strength \( f'_{m} \)
\( \varepsilon_{me} \) extreme fibre strain in wall section due to \( M_c \)
\( \varepsilon_{mi} \) elastic masonry strain immediately after prestressing
\( \varepsilon_{mp} \) masonry axial strain at initiation of post-peak strength plateau for Priestley-Elder stress-strain curve
\( \varepsilon_{mu} \) maximum dependable masonry strain
\( \varepsilon_{pu} \) ultimate elongation strain of prestressing steel
\( \varepsilon_{sh} \) concrete masonry final shrinkage strain
\( \phi \) curvature at wall section due to applied moment \( M \)
\( \phi \) wall average curvature in the plastic deformation zone at ultimate displacement capacity
\( \phi \)  
strength reduction factor

\( \phi_{cr} \)  
curvature at wall section due to \( M_{cr} \)

\( \phi_e \)  
curvature at wall section due to \( M_e \)

\( \phi_f \)  
flexural strength reduction factor

\( \gamma \)  
non dimensional crack length at \( M_e \)

\( \gamma_{0} \)  
drift demand at equivalent SDOF structure height

\( \gamma_{cr} \)  
curvature at wall section due to \( M \).

\( \gamma_i \)  
interstorey drift demand

\( \gamma_{i,max} \)  
interstorey drift limitation

\( \gamma_{u} \)  
wall ultimate drift capacity

\( \gamma_{\nu max} \)  
wall drift corresponding to \( d_{\nu max} \)

\( \gamma_{\nu} \)  
roof drift demand

\( \Lambda \)  
Loss ratio between prestress loss calculated with additive and incremental methods

\( \mu_d \)  
displacement ductility demand

\( v \)  
poisson’s ratio

\( \theta \)  
wall rocking rotation

\( \rho_a \)  
transverse confining ratio for Priestley-Elder stress-strain curve

\( \rho_v \)  
volumetric confining ratio for Priestley-Elder stress-strain curve

\( \omega_v \)  
seismic dynamic base shear amplification factor

\( x \)  
ratio of net concrete masonry unit area to total area of masonry unit and void

\( \xi \)  
wall axial load ratio

\( \xi_{\nu} \)  
viscous damping ratio

\( \xi_{\nu u} \)  
wall axial load ratio at \( M_u \)

\( \xi_{\nu w} \)  
wall axial load ratio at wall ultimate displacement capacity

\( \{ \phi_n \} \)  
mode shape vector for \( n^{th} \) mode
INTRODUCTION

1. PRINCIPLE OF UNBONDED POST-TENSIONED WALLS

Using unbonded post-tensioning, walls are vertically prestressed by means of strands or bars which are passed through vertical ducts inside the walls. As the walls are subjected to lateral displacements (in-plane forces), gaps form at the horizontal joints, reducing the system stiffness. As long as the prestressing strands are kept within the elastic limit, or at least maintain a considerable amount of the initial prestressing force, they can provide a restoring force which will return the walls to their original alignment upon unloading. Thus, the lateral force-displacement response may be idealised by a non-linear elastic relationship. The integrity of the walls is maintained as no tensile strains form in the wall units and there are no residual post-earthquake displacements.

The key feature in this behaviour is attributable to the tendons being unbonded over the entire wall height, allowing for distribution of tendon strain over the entire length of the tendon. This results in a large tendon elongation capacity before yielding which directly results in a large elastic deformation capacity of the wall panel. It is important to use high strength prestressing steel ($f_{py} > 900$ MPa) because of its larger elastic strain capacity when compared to ordinary steel grades (300-500 MPa). High strength prestressing steel improves the elastic wall displacement capacity and reduces the potential prestressing force loss due to creep and shrinkage. A single large gap opening is expected to occur at the base of unbonded post tensioned concrete walls, this location being a construction joint and subjected to the largest flexural demand.

Fig. 1.1 shows two Post-tensioned Concrete Masonry (PCM) wall solutions schematically. Both consist of a foundation beam, the masonry wall panel built of hollow core concrete blocks, and a beam (bond beam) on top of the wall accommodating prestressing anchorages. Typically grouting of the wall cavities is required. When grouting is required, prestressing
ducts are placed in the wall cavities during construction. Solution (a) is prestressed with flexible high tensile strength strands looping through the foundation beam, while solution (b) is prestressed with high tensile strength bars anchored in the foundation beam. Both solutions are post-tensioned, i.e. the prestressing is applied after construction of the wall. The prestressing force is normally applied using a hydraulic jack.

1.1 MOTIVATION FOR PCM RESEARCH

Prestressed concrete masonry can be regarded as a new construction form. The individual components, concrete masonry blocks and prestressing steel, are well known and widely available, however the combination of the two brings a new construction form with a set of unique properties amalgamating the compression strength of masonry and the tensile strength of the prestressing system. Through the pre-compression provided by the prestressing, cracking under serviceability loading may readily be eliminated. Effectively one has a material with efficiency similar to prestressed concrete.

In the last three decades prestressing has become widely used in the construction industry for buildings, e.g. for floor beams and floor slabs, and for other types of structures, notably bridges and offshore platforms. This extensive usage has led to the development of a variety of competing prestressing systems with reasonable pricing.

Post-tensioning of masonry has had limited application in New Zealand. As will be expanded on in the following chapter, there are currently no specific code requirements for the use of prestressed masonry in New Zealand. As a consequence of little knowledge about the ductility and energy dissipation characteristics of prestressed concrete masonry, among other factors, this
technology is currently not applied for seismic design in New Zealand. Designers are effectively discouraged from specifying prestressed masonry, partially due to uncertainty about the material and partially because of the liability risk.

The lack of detail on the use of prestressed concrete masonry in the current New Zealand masonry design standard is, in fact, due to a worldwide lack of understanding of the in-plane seismic behaviour of PCM, which differs considerably from the behaviour of reinforced masonry. There is a distinct lack of experimental data illustrated by the fact that the tests on PCM shear wall subjected to cyclic in-plane loading, as reported in this document, constitute the only testing of this kind yet to be carried out worldwide. Seismic design principles for PCM, therefore, need to be developed through a research programme. Theoretical concepts need to be refined and subsequently verified by laboratory experiments.

Recently, considerable attention has been given to the use of post-tensioned concrete walls. This research effort underlines the large potential of using post-tensioning in conjunction with concrete masonry walls because concrete masonry material properties are similar to those of concrete.

Motivation for the research reported herein derives from the fact that (1) the understanding of PCM walls subjected to seismic loading is limited, (2) the seismic structural performance is likely to be satisfactory (or better) (3) little structural testing has been conducted, and (4) PCM presents a potential industry opportunity.

In the wake of the above findings, an industry supported research programme on Prestressed Concrete Masonry (PCM) was initiated in 1997 at the University of Auckland. Initial research was funded by the University of Auckland and by the Cement and Concrete Association of New Zealand (CCANZ). The research activities in the past three years were primarily funded by the New Zealand Concrete Masonry Association (NZCMA) through a student grant.

1.2 SCOPE AND ORGANISATION OF THESIS

Scope:
The principal scope of the thesis is to provide detailed guidelines for complete analysis and design of ductile post-tensioned concrete masonry walls, including assessment of seismic demand and time dependent effects.
The focus in this thesis is on in-plane response of post-tensioned concrete masonry walls with un-bonded tendons and fully grouted wall cavities. Three structural testing series were conducted to create an experimental data base. Analytical procedures were developed for prediction and design of PCM walls subjected to lateral in-plane seismic forces. Material properties of concrete masonry were investigated in relation to strength, strain capacity and time dependent losses.

Organisation

The thesis is organised in the following main subjects:

*Literature Review:* Chapter 2 presents a literature review of the current state of research and codification of prestressed masonry. It focuses on concrete masonry because it is the dominant structural masonry material in New Zealand. Issues associated with materials and structural response are discussed. The conclusions identify the areas where research is needed, and serve as background and justification of the current prestressed concrete masonry research activities at the University of Auckland.

*Material Properties:* The objective of Chapter 3 is to discuss appropriate material properties for grouted concrete masonry manufactured in New Zealand, with particular emphasis given to the use of these parameters when designing prestressed concrete masonry (PCM) walls. The discussion targets concrete masonry crushing strength, modulus of elasticity and strain capacity, and assesses criteria from current international research and masonry design standards.

*Time dependent effects:* Assessment of prestressing losses for prestressed concrete masonry structures is presented in Chapter 4. The effective prestress level decreases over time due to creep, shrinkage and steel relaxation, hence reducing structural efficiency. A detailed review of the current state of research on the topic is presented and recommendations are given for New Zealand conditions with support from experimental results.

*Structural Testing:* Three series of structural testing were carried out. Chapters 5, 6 and 7 are concerned with interpretation of the testing results. The principal intent with these in-plane wall tests was to validate the use of PCM in a realistic structural configuration. Furthermore, the tests explored means of masonry confinement or strengthening that are expected to increase the reliable wall drift capacities.
Analysis and design: Chapter 8 is concerned with analytical methods for predicting the in-plane force-displacement relationship for unbonded post-tensioned concrete masonry. The scope of such methods is to describe the wall behaviour throughout the entire loading range, including non-linear large displacement response that can be anticipated in a seismic event.

Dynamic analysis: Chapter 9 is concerned with non-linear time-history analysis of unbonded post-tensioned concrete masonry (PCM) cantilever walls. Analyses were carried out in order to clarify the relationship between wall stiffness and strength, and the wall displacement demand due to dynamic seismic excitation. Pseudo displacement spectra ($S_u$) were developed for design use, and a displacement focused design procedure is outlined.
LITERATURE REVIEW

2. OVERVIEW

Post-tensioning of masonry has had limited application in New Zealand. There are currently no specific code requirements for the use of prestressed masonry, largely as a consequence of little knowledge about the ductility capacity and energy dissipation of prestressed concrete masonry, among other factors. Designers are effectively discouraged from specifying prestressed masonry, partially due to uncertainty about the construction form and partially because of the liability risk.

There are, however, a few examples of the use of prestressed concrete masonry in New Zealand. The classic example is a 5-storey building erected in Christchurch in 1970. The apartment complex was supported by 12 in. cavity walls with vertical prestressing strands [2-1]. At that time the solution was found to be both structurally and economically feasible. It should be noted that the design requirements at that time were less stringent. The earthquake loading was lower than seen today and the building was designed according to the allowable stress limit state.

In the United Kingdom prestressing of masonry is mostly used for wall and pier designs. The walls are primarily designed to resist out-of-plane loading, i.e. face loading such as strong winds or earth pressure. The in-plane forces in non-seismic areas are normally negligible in comparison to the face loading. Post-tensioning of masonry has typically been used for structures of unusual dimensions, e.g. for large masonry beam spans and tall walls. There are, for example, many reports on the use of prestressed masonry fin-walls [2-2] with eccentrically placed post-tensioning because of unidirectional loading applied to such structures as storage tanks and retaining walls. The fins (or pilasters) greatly increase the lateral strength of the wall, allowing for thin wall sections between the fins.
The use of prestressed masonry in the United States of America has been limited so far. There are reports on some two-storey homes, factory buildings, retaining walls and sound walls, e.g. [2-3, 2-4]. All of these were in non-seismic areas. Currently in the U.S.A., prestressed concrete masonry is designed in accordance with allowable stress design principles based on elastic response to seismic excitation. Therefore, prestressed masonry is presently not applied in seismic zones where design loads potentially can be reduced significantly by means of ductility and energy dissipation.

Post-tensioning can be used advantageously for rehabilitation of old buildings. Such application used for seismic strengthening is discussed by Ganz [2-5] who provides several case studies on masonry structures located on the American west coast. It is beyond the scope of this thesis to discuss the use of post-tensioning for masonry building rehabilitation, so no further details are given here on that subject.

Detailed discussions of the development of prestressed masonry research and application can be found in Schultz and Scolforo [2-6] and Lissel et al. [2-7].

2.1 CODIFICATION OF PRESTRESSED MASONRY

The development of prestressed masonry was pioneered in the United Kingdom [2-6], beginning in the late 1950’s and primarily promoted by daring structural engineers. The combined efforts of development through the years led to a draft code in the late 1970’s, which was eventually approved and incorporated into the general masonry code BS 5628, 1985: ‘Code of Practice for Use of Masonry’. This code is based on limit state design principles. The early uses of prestressed masonry predominantly involved clay brick, which is widely available in the U.K., but more recent efforts have incorporated prestressed concrete masonry. BS 5628 part 2, “Code of Practice for Use of Masonry, Structural Use of Reinforced and Prestressed Masonry” was revised in 1995 [2-8].

In the U.S.A., masonry codes have recently incorporated provisions for design of prestressed masonry. The process of developing the appropriate chapters for amendment to the codes had been proceeding since the early 1990’s. The proposals were explicitly recognised with the publication of the MSJC ‘Building Code Requirements for Masonry Structures’ in 2002 [2-9]. These were inspired by the relevant masonry codes and prestressing codes, and to some extent by BS 5628. The MSJC 2002 is partially based on the allowable stress concept, which simplifies calculations but also produces unnecessarily conservative results. MSJC recognises the use
of prestressed masonry for ductile seismic design. A minimum amount flexural reinforcement (mild steel) is required. Bonded prestressing steel may count towards the minimum amount of reinforcement but unbonded prestressing steel may not. This means that mild steel is required in the plastic hinge zone for unbonded post-tensioned masonry walls.

Prestressed masonry was first adopted in the Australian masonry design code with the issue of AS 3700-1998 [2-10]. This code mainly targets construction in non-seismic regions, but can be applied for design of structures in earthquake zones given design to the elastic response of the seismic forces. The prescriptions for prestressed masonry are largely inspired by BS 5628:1995 [2-8].

There are currently no specific code requirements for the use of prestressed masonry in New Zealand. In the New Zealand masonry design code NZS 4230:1990 [2-11] reference is made to the design of prestressed masonry, being conceptually parallel to design of prestressed concrete, but no guidance is given. However, the fact that prestressing of masonry is incorporated in the Australian masonry design standard is of significant importance in the New Zealand context due to the expected harmonisation of the New Zealand and Australian codes in the future. It seems natural that the design concepts for prestressed masonry in seismic zones for a future common masonry code should be developed in New Zealand, primarily because of New Zealand expertise in seismic design and because of less concern about earthquakes in Australia. This is clearly a strong incentive to push forward the research into prestressed masonry in New Zealand.

There has been significant interest in developing codes in many other countries. In Canada, for example, code development is well advanced and is coordinated with the U.S. effort.

2.2 CURRENT STATE OF RESEARCH

This chapter focuses on concrete masonry because it is the dominant structural masonry material in New Zealand. Fired brick is widely used for wall cladding, but rarely for load bearing purposes. In order to describe the current state of research, issues associated with materials and structural response are discussed. The subsequent conclusions identify the areas where research is needed, and serve as background and justification of the current prestressed concrete masonry research activities at the University of Auckland.
2.2.1 Materials

2.2.1.1 Concrete masonry

The strength and constitutive properties of grouted concrete masonry are well established as documented in Chapter 3. With the introduction of pre-compression, the creep and shrinkage parameters of concrete masonry become more important, and the higher magnitude of unrecoverable deformation over time leads to significant prestressing losses. There have been several studies on creep and shrinkage of concrete masonry, most recently by Hamilton and Badger [2-12] who studied U.S. concrete masonry. It is unclear whether these studies apply directly to the New Zealand type of concrete block, which may have significantly different creep and shrinkage behaviour due to its unique aggregate composition. The studies by Hamilton and Badger did not include experiments on fully grouted concrete masonry.

2.2.1.2 Prestressing steel

The constitutive properties of high strength prestressing steel are well established as documented in Chapter 3. In terms of time dependent effects, steel relaxation for high-strength steel also affects the effective prestress level over time. This depends on the type of high-strength steel, generally categorised as high relaxation or low relaxation. The steel relaxation effect is well defined and may be extracted from most codes for prestressed concrete, e.g. NZS 3101 [2-13].

2.2.2 Structural behaviour

2.2.2.1 Out-of-plane response

Current codes [2-8,2-9,2-10] that incorporate criteria on prestressed masonry, apply directly to non-seismic application where out-of-plane lateral forces typically govern design. For non-seismic application, these codes, supported by extensive laboratory research, appear to perform satisfactorily despite some differences. A selected list of out-of-plane wall tests can be found in [2-6] and [2-7].

An extensive literature review suggests that no tests of prestressed concrete masonry walls subjected to simulated out-of-plane seismic loading have yet been conducted. This lack of test data limits the knowledge about the ductility capacity and energy dissipation properties of PCM out-of-plane response, and necessarily leads to the use of design forces based on elastic
response to ground motion for design of PCM walls carrying significant out-of-plane seismic forces. Thus no force reduction due to ductility is permitted.

In most New Zealand applications, walls do not carry significant out-of-plane lateral seismic forces, other than those arising from wall self weight and the imposed overall structural displacements (seismic forces are resisted by other structural elements). In that case the requirement for ductility and energy dissipation will be low and design to the elastic seismic response will probably be satisfactory.

2.2.2.2 In-plane response

The in-plane response of prestressed masonry walls for non-seismic design has been studied in some detail. Page and Huizer [2-14,2-15] and Huizer and Shrive [2-16] have reported on monotonic lateral in-plane load tests of post-tensioned hollow clay masonry shear walls.

Page and Huizer [2-14] tested three clay masonry walls. The walls were 3.0 m high and 2.5 m long, constructed in running bond of hollow clay masonry units. The walls were reinforced vertically with four VSL prestressing bars. Walls A and B had the four VSL bars prestressed vertically to approximately 390 kN upon construction. These walls were not grouted. The VSL bars in wall C remained unstressed but were bonded to the wall by grouting of the wall cavities. Furthermore wall A had four VSL bars embedded horizontally which were post-tensioned to a total of about 200 kN. Testing of the walls in a monotonic push excursion revealed significantly different behaviour. The maximum applied horizontal strength was 146 kN, 175 kN and 115 kN for walls A, B and C, respectively. Wall A failed prematurely due to a construction error but the actual wall strength was assessed to be significantly higher than 175 kN. The typical failure mode for all walls was tensile splitting of the diagonal compression strut. Application of prestress demonstrated that the vertical precompression in wall B and more so the vertical and horizontal precompression in wall A deferred wall tensile splitting to higher horizontal loads when compared to the non-prestressed wall C. Effectively larger strength and displacement were reached for wall B (and also for wall A had it not failed prematurely) during testing. These wall tests attest to the significant increase in masonry shear strength due to prestressing. However, they did not provide much information on prestressed wall behaviour in the context of ductile seismic design where reversing displacement excursions far beyond the wall yield displacement are expected. Similarly, the tests reported in [2-15] and [2-16] demon-
strated that prestressed masonry walls provide higher strength and comparable deformation capacity, when compared to reinforced masonry walls.

The design procedures for shear walls subjected to in-plane forces, suggested by [2-14], [2-15] and [2-16], largely follow those for the out-of-plane response as given by [2-8,2-9,2-10]. These codes have drawn on research into both prestressed concrete and prestressed concrete masonry [2-6]. The procedures treat prestressed masonry walls as conceptually similar to unreinforced masonry walls with the prestress acting as an additional externally applied axial load.

Currently, limited experimental data exists for prestressed concrete masonry walls under in-plane seismic loading. It is in fact believed, that the only tests to have considered unbonded post-tensioned concrete masonry walls subjected in-plane seismic loading are those conducted at the University of Auckland, as reported in Chapters 5, 6 and 7. For proper in-plane ductile seismic design of unbonded post-tensioned concrete masonry walls, it is imperative to understand the ultimate behaviour of prestressed masonry. It is important to be able to categorise various failure modes, such as flexural failure characterised by concrete compression failure or steel rupture, shear failure characterised by diagonal tension cracks or bed joint sliding, joint failure and axial instability. As knowledge of these modes of failure, and their associated strength and ductility capacity is heavily dependent on experimental work, it is a cumbersome process to establish an appropriate design procedure. The experimental database must consider cyclic load tests, simulating seismic motion. Clearly more in-plane testing is needed.

On the contrary, there has been extensive research undertaken on other types of prestressed systems under seismic loading, notably prestressed concrete. The culmination of this research was testing of a precast prestressed concrete wall with unbonded tendons carried out by the “PRESSS” coordinated research group in the USA [2-17,2-18]. The elevation of the large scale wall is shown in Fig. 2.1. It consisted of four panels each 9’ (2.74 m) long, 18’-9” (5.72 m) tall, 1’ (0.305 m) thick, and stacked in a 2 x 2 pattern. The wall was prestressed vertically with unbonded prestressing tendons that were located centrally in the wall panels. The wall panels were clamped together vertically by the prestressing such that panels 1 and 3 (P13) could move independently in the vertical direction from panels 2 and 4 (P24). UFP connectors (energy dissipators) were located in the interface between P13 and P24 such that relative vertical displacement between P13 and P24 due to wall lateral displacement would force the UFP connectors to dissipate energy. The wall panels were lightly reinforced vertically. Panels 1 and 2 had special confinement reinforcement embedded in the compression toes.
Applying cyclic lateral load to the wall showed that high displacement capacity could be achieved, far beyond 2% drift, and that this displacement capacity relied on integrity of the wall toe regions which acted as pivot points for the observed rocking behaviour. The onset of rocking could be compared to a reinforced concrete wall reaching its yield strength. After onset of rocking the wall strength only increased at a low rate as little stress increase occurred in the unbonded prestressing tendons as a result of the lateral displacement. Failure never occurred to the wall despite being taken to drift ratios far beyond 2% and only damage of cosmetic character in the compression toe regions occurred.

A similar study was conducted at the University of Canterbury [2-19,2-20].

In conclusion, testing of unbonded post-tensioned concrete walls proved that extremely satisfactory in-plane behaviour could be achieved with the prestressing tendons remaining elastic for interstorey drifts of the order of 2%. Only damage of cosmetic character occurred to the confined corners of the walls. The PRESSS study also found that the displacement response

---

Fig. 2.1—Schematic of PRESSS wall test [2-17]
amplitude could be effectively controlled (limited) by incorporation of energy dissipators. Insignificant residual wall displacement was observed after sustaining the peak drift. The structural behaviour of this type of wall resembles that of PCM, so results are highly relevant to the current study.

2.2.2.3 Joints

Wall to floor joints could potentially be simplified with the use of prestressing. As suggested by Ganz [2-21], a system with precast concrete floor slabs clamped between prestressed concrete masonry walls from the below and above storeys could potentially reduce the amount of mild steel in the joint. Reinforcement congestion in the joint can be avoided and speed of construction improved. There has been some research into this type of joint for concrete solutions for non-seismic application. There is, however, a total lack of experimental data for prestressed concrete masonry joints of this type when subjected to seismic loading.

2.2.2.4 Prestress loss

Guidelines for assessing the prestress loss are provided by existing masonry codes, e.g. BS 5628 [2-8], MSJC [2-9] and AS 3700 [2-10]. Application of the provisions in these codes to similar materials results in considerable difference in the calculated prestress losses. It has been estimated [2-6] that the prestressing losses over time due to creep, shrinkage, steel relaxation etc. for concrete masonry may amount to up to 30% of the initial tendon stress.

2.3 CONCLUSIONS

The following conclusions on the current state of prestressed concrete masonry wall research are drawn from the above discussion. These conclusions focus on the use of unbonded post-tensioning, which is the topic of this thesis. Furthermore, the focus of this document has been limited to studying prestress loss due to time dependent effects and in-plane wall response.

(1) Material properties for concrete masonry and prestressing steel, in terms of constitutive behaviour (stress-strain relationship), have been researched extensively and are relatively well understood.

(2) There is a need for better understanding of the creep and shrinkage properties for concrete masonry subjected to axial loading, with a view to New Zealand conditions. This is particularly the case for fully grouted concrete masonry, which has not been thoroughly examined experimentally.
Numerous laboratory tests have been conducted on the out-of-plane response of pre-stressed concrete masonry walls, but only few in-plane tests. Little research has been done on in-plane seismic response, neither experimentally nor theoretically, in particular for the case of unbonded post-tensioned concrete masonry walls.
2.4 REFERENCES


Chapter 3

MATERIAL PROPERTIES

3. OVERVIEW

The objective of this chapter is to discuss appropriate material properties for grouted concrete masonry manufactured in New Zealand, with particular emphasis given to the use of these parameters when designing prestressed concrete masonry (PCM) walls. The discussion targets the concrete masonry crushing strength, modulus of elasticity and strain capacity, seen from a New Zealand perspective. However, a review of criteria from current international research and masonry design standards is included. Strength and strain enhancement of the wall compression zone due to confinement is also discussed, and a concrete masonry constitutive relationship model for detailed structural analysis is described and evaluated. Prestressing steel material properties are discussed at the end of this chapter. Time dependent effects in PCM, notably masonry creep and shrinkage, and prestressing steel relaxation are discussed in Chapter 4.

It is noted that the term 'PCM' in this chapter refers to post-tensioned concrete masonry walls with unbonded tendons, suitable for ductile seismic design.

Design of PCM generally benefits from an accurate estimation of the material properties. Currently, the New Zealand standard for design of masonry structures NZS 4230:1990 [3-1] specifies unrealistically low masonry compression strength. This may not greatly affect the design of reinforced masonry, but certainly affects the design of PCM, where increased axial wall forces place greater demand on the available compression strength of the masonry. Similarly, the current New Zealand practice is to adopt unrealistically high values of the masonry elastic modulus, thereby underestimating the structural period and overestimating the design lateral forces resulting from earthquake response. Structural testing of PCM walls has shown large masonry strain capacity of the flexural compression zone. Strain values far beyond those specified in NZS 4230 have consistently been measured, suggesting that current code criteria are unnecessarily disadvantageous when designing PCM. It is argued that there is little value in
using either an unrealistically high or unrealistically low assessment of material properties when conducting structural design, and that attention should instead be given to formulation of accurate predictive methods to establish characteristic material properties.

3.1 STRESS-STRAIN CHARACTERISTICS

Grouted concrete masonry is a composite material consisting of hollow precast concrete blocks laid up with horizontal and vertical mortar joints and subsequently filled with grout. The properties of such material consequently vary considerably depending on the proportions and geometry of the ingredients, and the quality of workmanship.

Fig. 3.1 shows typical stress-strain relationship curves for grouted concrete masonry subjected to uniaxial compression [3-2]. Three curves are shown: one for plain grouted masonry (unconfined) and two for grouted masonry with confining plates in the bed joints. Four distinct parameters describe the material behaviour: (1) the initial elastic modulus \( E_m \), (2) the peak strength \( f' m \), and (3) its associated strain capacity \( \varepsilon_m \), and (4) the ultimate strain capacity \( \varepsilon_{mu} \). Plain grouted masonry (unconfined) typically reaches the peak strength at a strain of 0.0015-0.002 and provides reliable strength at strains up to about 0.003-0.004.

Confining the prisms with plates in the horizontal bed joint improves masonry performance dramatically [3-2]. The peak strength is improved, but most importantly the ultimate strain capacity, \( \varepsilon_{mu} \), is increased significantly to provide reliable strength at strains of up to 0.008-0.010 for 190 mm high masonry blocks confined in each bed joint and higher for 90 mm high masonry blocks confined at each bed joint.
3.2 CRUSHING STRENGTH

Meaningful estimation of the masonry compression strength, $f'_m$, is important for accurate prediction of PCM wall behaviour. Fig. 3.2 shows the predicted in-plane lateral force-displacement (F-D) relationship for a realistic PCM wall, 3.6 m long, 15 m high and 190 mm thick, post-tensioned with four 15 mm high strength prestressing strands. A self-weight of 200 kN, dead and live load of above floors of 665 kN and an initial prestress of 70% of the yield stress were assumed. The force-displacement relationships were predicted based on the analytical method presented in Chapter 8 for an effective height, $h_0$, of 10 m using confined masonry (CP100) compression strengths, $f'_m$ of 8 MPa, 12 MPa, 18 MPa and 24 MPa, with all other parameters the same. It is seen that the wall strength increases significantly with an increase of $f'_m$, as a direct result of compression zone length shortening and an associated increase of lever arm between the flexural compression zone centroid and the centroid of the wall axial load. Fig. 3.2 also shows that the wall displacement capacity at nominal flexural strength (symbol Δ) and the ultimate displacement capacity (symbol O) increase with an increase of $f'_m$. This effect is caused by shortening of the compression zone, allowing higher curvature and rotation of the hinging zone given the extreme fibre strain associated with the respective limit state. The example shown in Fig. 3.2 suggests that the effective wall stiffness at maximum serviceability moment (extreme fibre compression of 0.55$f'_m$) (symbol ×) reduces with an increase of $f'_m$ because of a shorter compression zone length and a longer wall base crack.

![Masonry compressive strength $f'_m$](image)

**Fig. 3.2—Typical PCM cantilever wall force-displacement characteristics**
The masonry compression strength, $f'_m$, must be assumed for design. The designer has the following options: (i) adopt a code-defined characteristic strength based on material choice and/or workmanship, (ii) specify higher strength justified by a record of consistent high strength construction provided by the contractor/mason or (iii) specify higher strength to be validated by testing of prism or grout cylinders made on site during construction.

Using options (i) or (ii) eliminates the need for material testing for the particular job. Option (ii) most likely will allow assumption of a higher strength than using (i), given a consistent level of workmanship. Option (iii) requires material testing specific to the job. Testing of masonry prisms is a cumbersome affair and concrete testing machines rarely are able to accommodate the prism dimensions. Testing of grout cylinders is much easier to handle.

In the New Zealand context, $f'_m$ is associated with the concrete masonry compressive strength found by testing of prisms with a minimum height-to-thickness ratio of 3. Typical NZ prisms are three blocks high and have height-to-thickness ratios of 4.3 and 3.2, for 15 series and 20 series masonry blocks respectively.

3.2.1 Grade dependent strength

Currently, the New Zealand masonry design code NZS 4230:1990 specifies the compressive strength for fully grouted masonry according to the level of supervision on the construction site. Three grade levels are stipulated as shown in Table 3.1. It is observed that the design strength is not related to the actual material strengths of blocks, mortar and grout, except for a minimum concrete block strength requirement of 12 MPa. The material specifications for mortar and grout are found in NZS 4210:2001 [3-3], that stipulates a minimum mortar strength of 12 MPa and a minimum grout strength of 17.5 MPa.

The New Zealand system of grade dependent strength is essentially of historical character and will most likely be replaced in the near future by a system based on characteristic strength of the actual materials, similar to the approach found in the Australian standard for masonry structures AS 3700 [3-4].

**TABLE 3.1—NZS 4230:1990 Masonry grades**

<table>
<thead>
<tr>
<th>Grade</th>
<th>Interpretation</th>
<th>Compressive strength $f'_m$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Continuously supervised construction</td>
<td>8 or higher when confirmed by testing</td>
</tr>
<tr>
<td>B</td>
<td>Supervised construction</td>
<td>8</td>
</tr>
<tr>
<td>C</td>
<td>Non-supervised construction</td>
<td>4</td>
</tr>
</tbody>
</table>
3.2.2 Characteristic strength

Modern manufacture of concrete products is associated with strict quality control. A statistical record of strength is required so that the most likely strength (average strength) and the characteristic strength (typically 5 percentile value) of the product can be established.

The New Zealand standard for manufacturing of typical hollow normal-weight concrete masonry blocks, NZS 3102:1983 [3-5], details that 49 out of every 50 blocks attain a strength of at least 12 MPa and that 45 out of every 50 blocks attain a strength of at least 14 MPa.

NZS 3109:1997 [3-6] details the minimum production standards for various concrete categories. Assuming high grade grout production with a performance record and the lowest available specified strength of 17.5 MPa, the statistical descriptors are target mean strength of 25 MPa and coefficient of variation of 15.5.

Assuming that both block and grout strengths are normal distributed random variables, one finds the following characteristics: (1) block mean strength = 17.3 MPa, standard deviation = 2.59 MPa, characteristic strength = 13.0 MPa (5 percentile) and (2) grout mean strength = 25 MPa, standard deviation = 3.94 MPa, characteristic strength = 18.5 MPa (5 percentile).

Priestley and Chai [3-7] found that the grouted masonry compressive strength for 190 mm high units with 10 mm mortar bed thickness could appropriately be calculated from knowledge of the block strength (uniaxial strength of one block), $f'_{cb}$, and grout strength (typically 100 mm diam. by 200 mm cylinder), $f'_{g}$, according to the following formula:

$$f'_{m} = 0.59x f'_{cb} + 0.90(1-x) f'_{g}$$

(3.1)

where $x$ is the ratio of the net concrete block area to the total area of the block and void, taken as $x = 0.45$ for 20 series blocks and 0.55 for 15 series blocks. Eqn. 3.1 was based on prism test data from New Zealand and North America. Assuming that the block and grout strengths are independent random variables, it can be shown that $f'_{m}$ is also a normal distributed random variable with mean strength of 17.0 MPa and standard deviation of 2.07 MPa for 20 series masonry, and mean strength of 15.7 MPa and standard deviation of 1.80 MPa for 15 series masonry. Fig. 3.3 illustrates the strength distribution of grout and mortar, and the resulting strength distribution for grouted 20 series concrete masonry. Characteristic masonry strengths $f'_{m}$(char) of 13.6 MPa and 12.8 MPa for 20 series and 15 series masonry respectively, are determined based on the 5 percentile value of its normal distribution; effectively values that
can safely be specified for supervised construction in New Zealand without any need for material testing.

Eqn. 3.1 also provides a method for verification of $f_m$ using the characteristic block strength provided by the manufacturer in conjunction with the characteristic grout strength found by cylinder testing. Using Eqn. 3.1 in this way, it was recommended [3-7] to apply a strength reduction factor of 0.75 to derive a lower-bound value.

3.2.3 Experimental data

The experimental data presented here was recorded in conjunction with PCM wall testing at the University of Auckland. Results from testing of unconfined and confined grouted concrete masonry were taken from wall test Series 1 and 2 described in Chapters 5 and 6.

Table 3.2 shows the average strength of unconfined masonry prisms, grout and mortar measured for 10 wall tests. Generally, three prisms, three mortar cylinders and three grout cylinders were tested for each wall. Prism construction was performed by registered masons using standard 140 mm precast grey hollow core concrete blocks that were provided by a range of manufacturers. The blocks were filled with standard 17.5 MPa target strength grout. Prior to grouting, SIKA CAVEX™ shrinkage compensating agent was added to the grout. All test cylinders were 100 mm diam. and 200 mm high.
It is observed from Table 3.2 that an average prism strength of at least 14.4 MPa was measured in all cases. An average prism strength of 18.1 MPa and a standard deviation of 2.3 MPa was found based on the total number of prisms tested. Assuming normal distribution of the strength, a 5 percentile strength for $f_{m}'$ of 14.3 MPa is found; thus the individual averages for each wall and the 5 percentile strength based on all wall tests exceeded the minimum characteristic design strength of 12.8 MPa derived from Eqn. 3.1 above. It is noticed that there is considerable scatter in the measured grout strength, ranging from 11.1 MPa to 35.0 MPa. It is of some concern that the measured grout strength in 4 out of 8 cases fell well below the code minimum specified strength of 17.5 MPa, even with consideration of the presence of the shrinkage compensating agent. The mortar strength, $f_{j}'$, also exhibited considerable scatter; however the measured minimum average of 10.5 MPa was in reasonably good agreement with the expected minimum strength of 12 MPa.

Interestingly, little scatter was measured in the prism tests, despite the large amount of scatter measured for both grout and mortar strengths. This may suggests that unconfined masonry strength is influenced more by the block strength than the grout and mortar strengths.

Table 3.3 shows results from testing of 9 prisms. All prisms were constructed of 15 series masonry blocks (140 mm thick) using 10 mm bed joints with blocks, mortar and grout from the same batches. The series consisted of 3 unconfined prisms with 190 mm block height (U200), 3 confined prisms with 190 mm block height (CP200) and 3 confined prisms with 90 mm block height (CP100). The confining plates were fabricated from 25 mm wide and 3.1 mm

<table>
<thead>
<tr>
<th>WALL TEST</th>
<th>$f_{m}'$ PRISM</th>
<th>$f_{g}'$ CYLINDER</th>
<th>$f_{j}'$ CYLINDER</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16.6</td>
<td>11.1</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>20.5</td>
<td>23.7</td>
<td>26.1</td>
</tr>
<tr>
<td>3</td>
<td>20.6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>14.4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>18.8</td>
<td>13.9</td>
<td>10.9</td>
</tr>
<tr>
<td>6</td>
<td>18.2</td>
<td>18.6</td>
<td>13.5</td>
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<tr>
<td>7</td>
<td>17.8</td>
<td>12.3</td>
<td>14.9</td>
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<tr>
<td>8</td>
<td>15.9</td>
<td>16.2</td>
<td>10.5</td>
</tr>
<tr>
<td>9</td>
<td>-</td>
<td>35.0</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>-</td>
<td>19.0</td>
<td>23.6</td>
</tr>
<tr>
<td>MEAN</td>
<td>18.1</td>
<td>17.5</td>
<td>14.6</td>
</tr>
<tr>
<td>ST. DEV.</td>
<td>2.3</td>
<td>8.0</td>
<td>5.6</td>
</tr>
</tbody>
</table>

MPa MPa MPa
thick mild steel. Fig. 3.4 provides details of nominal geometry and specifications of the prisms and confining plates. Compressive strains were measured with four displacement transducers attached to the prisms using a gauge length of 400 mm. It is seen from Table 3.3 that the confining plates did not increase the masonry crushing strength considerably.

Fig. 3.4—Prism and confining plate dimensions and specification

<table>
<thead>
<tr>
<th>Table 3.3—Confined prism testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prism</td>
</tr>
<tr>
<td>---------</td>
</tr>
<tr>
<td>U200</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>mean</td>
</tr>
<tr>
<td>st. dev.</td>
</tr>
<tr>
<td>CP200</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>mean</td>
</tr>
<tr>
<td>st. dev.</td>
</tr>
<tr>
<td>CP100</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>mean</td>
</tr>
<tr>
<td>st. dev.</td>
</tr>
</tbody>
</table>

Mortar strength: 14.9 MPa; Grout strength 12.3 MPa, *strain at peak force, **strain at approx. 50% loss of strength, #based on slope in 0.05-0.33$f_m$ interval
3.2.4 Australian approach

According to the Australian masonry standard AS 3700-1998 [3-4], the designer has two options for specifying the masonry design strength: (1) use \( f'_{m} \) derived on a statistical basis and related to the utilised block type or (2) derive \( f'_{m} \) from prism testing.

AS 3700 associates \( f'_{m} \) with the ‘Unconfined Compressive Strength’, in the Australian context meaning the compressive strength found by testing of slender prisms with a height-to-thickness ratio of 5. Prisms of such proportions are deemed unaffected by the confining effect of the testing machine platens. Compressive strength for prisms of lower height-to-thickness ratio are converted to the ‘Unconfined Compressive Strength’ by means of code defined aspect ratio factors (Appendix C in AS 3700). Note that the expression ‘unconfined’ refers to plain grouted concrete masonry (no confining plates embedded) in the general context of this thesis; only in the context of AS 3700 is the expression related to testing of slender prisms.

Statistical base: Knowing the characteristic unconfined strength of the masonry units, \( f'_{uc} \) and the bedding type, AS 3700 section 3.3.2 is applied to determine the characteristic strength (5 percentile) for ungrouted concrete masonry. The manufacturer generally provides \( f'_{uc} \) based on continuous sampling. There is no direct dependency on the grout properties, however it is specified that the characteristic grout strength, \( f'_{cg} \) must be higher than 12 MPa and lower than \( 1.3f'_{uc} \). Using \( f'_{uc} \) of 15 MPa, a level of strength often encountered in New Zealand, and typical concrete masonry dimensions, \( f'_{m} = 8.0 \) MPa is found. Using AS 3700 section 7.3.2, defining the axial strength of grouted masonry based on \( f'_{m} \) and \( f'_{cg} \), a grouted concrete masonry strength of 6.0 MPa was calculated. That strength falls short of both the minimum characteristic strength for NZ masonry derived above of 12.8 MPa and the NZS 4230 grade B masonry strength of 8 MPa. It is noted that the aspect ratio factor defined by AS 3700 is approximately 0.9 for typical NZ prism dimensions, thus not explaining the low AS 3700 compressive strength estimate.

Prism testing can be used to establish the characteristic 5 percentile strength, \( f'_{m} \). It appears that prism sampling can be done by the contractor/mason and that the sampling does not have to be directly associated with the project in mind. It is also stated that for wall strength verification, i.e. testing of prisms made on site during construction, the testing average strength should be used for verification. In that case, with only a few prisms tested, the most likely strength (the average) is used instead of the characteristic strength (5 percentile).
3.2.5 USA approach

The Masonry Standards Joint Committee, constituted by the American Concrete Institute (ACI), the American Society of Civil Engineers (ASCE) and The Masonry Society (TMS), proposes an approach [3-8] for specification of the masonry compression strength similar to that of AS 3700. There are two options: (1) to specify $f'_m$ from tabulated values that depend solely on the concrete block strength and the mortar type (Table 2, page S-10). It is required that the grout strength at least equals $f'_m$ and exceeds 13.8 MPa; (2) $f'_m$ may be determined from prism testing. Specifying mortar type N and a characteristic block strength of 15 MPa results in a design compression strength of 10.3 MPa. That strength exceeds the tabulated predictions of both NZS 4230 and AS 3700, but falls short of the minimum characteristic strength of 12.8 MPa for NZ grouted concrete masonry.

3.3 ELASTIC MODULUS

In the context of seismic design, it is important to accurately estimate the structural stiffness. This requires precise knowledge of the elastic modulus, $E_m$, of the masonry.

A high estimate of $E_m$ results in a low estimate of the fundamental period of vibration and therefore generally higher seismic design loading, while the corresponding structural displacements are underestimated. The result is a conservative estimate of seismic loading but an unconservative estimate of structural displacements. A low estimate of $E_m$ results in the opposite effect, i.e. overestimation of structural displacement and underestimation of the seismic actions. Not only does $E_m$ influence the flexural response, it also governs the prediction of shear deformation.

Unbonded PCM walls subjected to in-plane seismic loads are expected to develop a single horizontal crack along the wall/foundation interface and no significant cracking above, suggesting elastic response of the panel above the hinging zone adjacent to the foundation. The stiffness of the uncracked PCM panel therefore depends almost solely on $E_m$, while e.g. a reinforced masonry panel with extensive cracking is less sensitive to $E_m$ because of the considerable influence of the reinforcing steel.

3.3.1 Current New Zealand codification

The current New Zealand masonry code NZS 4230:1990 simply specifies that $E_m = 25$ GPa, irrespective of masonry characteristics, unless established otherwise by testing. In reality this
clause overestimates $E_m$ almost by a factor of two for grouted concrete masonry. The value was set this high deliberately to ensure conservative estimates of the seismic design forces. In addition, little emphasis was placed on accurate calculation of displacements at the time of writing of the code.

3.3.2 Literature and other codes

Extensive research has been carried out on establishing appropriate expressions for $E_m$. It was found that the elastic modulus is not clearly related to any property of block, mortar, grout or prism dimensions, but influenced by all of these. For simplicity $E_m$ is generally related to $f'_m$ because the crushing strength is also influenced by these parameters. Examples of recommendations and code equations are:

$$E_m = 900f'_m \quad \text{AS 3700-1998 [3-4]} \quad (3.2)$$
$$E_m = 900f'_m \quad \text{MSJC 2002 [3-8]} \quad (3.3)$$
$$E_m = 1000f'_m \quad \text{Paulay and Priestley [3-9]} \quad (3.4)$$
$$E_m = 750f'_m \quad \text{UBC 1997 [3-10]} \quad (3.5)$$
$$E_m = 750f'_m \quad \text{NEHRP 1997 [3-11]} \quad (3.6)$$
$$E_m = 850f'_m \quad \text{CSA S304.1-M94 [3-12]} \quad (3.7)$$

The above formulae were generally developed to reflect the elastic modulus in relation to measured prism strength. It is thus incorrect to use the specified masonry strength (5 percentile) for evaluation of $E_m$ because the strength likely to be encountered (mean strength) is expected to exceed the specified strength. A typical 5 percentile strength to mean strength ratio is 80%. It is also mentioned that the formulation of $E_m$ is sensitive to the testing procedure, i.e. the stress range on which the elastic modulus is based. The typical stress range is 0.05-0.33$f'_m$.

3.3.3 Experimental data

The experimental data shown in Table 3.3 represent typical values of $E_m$ for grouted concrete masonry in the Auckland region. The results, measured in the stress range of 0.05-0.33$f'_m$, suggest little effect of confinement on the $E_m/f'_m$ ratio. Based on the above equations and the experimental data in Table 3.3, the following relationship based on the actual crushing strength is proposed for New Zealand conditions:
\[ E_m = 800f'_m \text{ (mean)} \]  \hspace{1cm} (3.8)

When using the characteristic value of \( f'_m \), it follows from the discussion in section 3.3.2 that the best estimate of the elastic modulus would be:

\[ E_m = 800f'_m \text{ (char)/0.8} = 1000f'_m \text{ (char)} \]  \hspace{1cm} (3.9)

### 3.4 MASONRY STRAIN CAPACITY

Accurate estimation of the ultimate masonry strain capacity is important for unbonded PCM walls that are expected to undergo large in-plane structural displacements due to seismic action. It is shown in Chapter 9 that structural displacement limitations are likely to govern ductile seismic design of PCM walls, e.g. limitations to the overall structural drift angle and interstorey drift angle.

The masonry crushing strain is highly influential on the wall lateral force-displacement characteristic. Increased strain capacity normally results in slightly higher wall strength due to shortening of the compression zone, but more importantly it most likely will promote higher displacement capacity because of larger curvature capacity of the hinging zone.

The definition of ultimate strain capacity related to flexural response can be differentiated from the definition related to uniaxial loading. It is clear that a uniaxial test unit reaches the maximum strength when all masonry fibres in the cross section attain \( \varepsilon_m \) simultaneously (refer to Fig. 3.1). Structural testing of PCM walls subjected to in-plane flexural loading generally indicated that the overall maximum wall strength was achieved for masonry strains in the extreme compression fibre beyond \( \varepsilon_m \). In fact the maximum wall strength was often associated with a peak strain larger than the code defined ultimate strain (often termed 'maximum usable strain') listed in Table 3.4. It is therefore argued that the ultimate masonry strain, \( \varepsilon_{mu} \), should be based on results from wall flexural testing.

#### 3.4.1 Code rules and recommendations

Table 3.4 summarises a series of design code requirements and recommendations for the ultimate masonry strain to be used for the design of reinforced concrete masonry. The values for unconfined masonry vary between 0.0025 and 0.0035, depending on the source, with the New Zealand codified value of 0.0025 as the lowest in the field. The two estimates of ultimate strain for confined masonry of 0.008 originate from the same research [3-2]. It is noted that the strain values given in Table 3.4 are related to flexural action and nominal flexural strength, and are...
therefore higher than the strain values associated with the masonry peak strength found by uniaxial compression testing, as suggested in Fig. 3.1 and Table 3.3. It appears that Australia and New Zealand should consider both moving to $\varepsilon_{\text{mu}} = 0.003$ for unconfined masonry, facilitating harmonisation and providing consistency with other international codes, and with the New Zealand concrete code NZS 3101:1995 [3-13].

### TABLE 3.3—Ultimate compression strain

<table>
<thead>
<tr>
<th>Source</th>
<th>Unconfined U200</th>
<th>Confined CP200</th>
</tr>
</thead>
<tbody>
<tr>
<td>NZS 4230:1990 [3-1]</td>
<td>0.0025</td>
<td>0.0080</td>
</tr>
<tr>
<td>AS 3700-1998 [3-4]</td>
<td>0.0035</td>
<td>–</td>
</tr>
<tr>
<td>MSJC 2002 [3-8]</td>
<td>0.0025</td>
<td>–</td>
</tr>
<tr>
<td>UBC 1997 [3-10]</td>
<td>0.0030</td>
<td>–</td>
</tr>
<tr>
<td>NEHRP 1997 [3-11]</td>
<td>0.0025</td>
<td>–</td>
</tr>
<tr>
<td>Paulay and Priestley [3-9]</td>
<td>0.0030</td>
<td>0.0080</td>
</tr>
</tbody>
</table>

3.4.2 PCM testing results

Table 3.5 shows results from testing of 6 unconfined PCM walls (refer to Chapter 5), reporting the vertical strain in the extreme fibre at nominal flexural strength $M_n$, at first observation of distress and at maximum strength, $V_{\text{max}}$. It is seen that the strains at nominal strength correspond well with the values in Table 3.4 with an average strain of about 0.003. At first distress (initiation of vertical splitting cracking of block face shell), the strain values had doubled and at $V_{\text{max}}$ the values had attained at least 0.01. It is noted that the values given in Table 3.5 were measured over the lowest 100 mm above the foundation, thus some degree of confinement of the first course of masonry was caused by the reinforced concrete foundation.

Confined masonry is expected to sustain even larger strains at maximum strength. Table 3.3 shows that uniaxial testing of CP100 resulted in a post-peak strength of 50% of $f'_{\text{m}}$ for $\varepsilon_{\text{mu}} =$

### TABLE 3.5—Vertical masonry strain, test results

<table>
<thead>
<tr>
<th></th>
<th>at nominal strength, $M_n$</th>
<th>at first distress</th>
<th>at maximum strength, $V_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0025</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>2</td>
<td>0.0027</td>
<td>0.0067</td>
<td>0.0137</td>
</tr>
<tr>
<td>3</td>
<td>0.0039</td>
<td>0.0104</td>
<td>0.0104</td>
</tr>
<tr>
<td>4</td>
<td>0.0027</td>
<td>0.0104</td>
<td>0.0104</td>
</tr>
<tr>
<td>5</td>
<td>0.0033</td>
<td>0.0074</td>
<td>0.0224</td>
</tr>
<tr>
<td>6</td>
<td>0.0031</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>strain</td>
<td>strain</td>
<td>strain</td>
</tr>
</tbody>
</table>

- 31 -
0.021 (mean); indeed a very large strain capacity for a concrete material. Results from testing of confined CP200 PCM walls showed strains measured over the lowest 100 mm of 0.01 to 0.02 at $V_{\text{max}}$ and testing of confined CP100 PCM walls showed average strains measured over the lowest 200 mm of 0.025 at $V_{\text{max}}$ (refer to Chapter 7). A discussion of maximum useable strain for ductile PCM wall design is given in Section 8.1.7.

### 3.5 FLEXURAL STRESS DISTRIBUTION

Despite general consensus on the definition of the uniaxial masonry compression strength, $f'_{\text{m}}$, notably when derived from prism testing, there is considerable variation of the definition of the stress distribution at nominal flexural strength. Fig. 3.5 shows three code definitions of the equivalent rectangular stress block for unconfined concrete masonry related to $f'_{\text{m}}$ and the neutral axis position, $c$. The MSJC stress block is specified for in-plane design, while no distinction between in-plane and out-of-plane design is made in NZS 4230 and AS 3700.

The use of larger rectangular stress block parameters leads to a reduced neutral axis depth, resulting in a larger flexural strength and a larger curvature capacity. The AS 3700 stress block, with a height of $1.3f'_{\text{m}}$, appears mainly to be associated with out-of-plane response. It is understood that the term $1.3f'_{\text{m}}$ stems from the Australian concrete design code provisions and reflects the conversion from 'Unconfined Compressive Strength' to test cylinder strength using
a height-to-thickness ratio of 2. Given the origin of this term, the suitability of the AS 3700 flexural stress block for limit state design of grouted concrete masonry is questionable. The American and New Zealand stress block parameters are comparable, and in reality predict similar behaviour. In-plane testing of PCM walls reported in Chapter 5 supports using the NZS 4230 or MSJC definition.

3.6 THEORETICAL STRESS-STRAIN RELATIONSHIP

Further to the above discussion, accurate modelling of the concrete masonry constitutive relationship is essential for detailed calculation of the global response of PCM walls with unbonded tendons. This material model should be of continuous nature to allow for capture of the wall response in the service load regime (elastic response), at nominal wall strength and at large displacement response, including possible strength degradation.

The NZ and US design code approaches for calculation of concrete masonry response are concerned with the elastic modulus and with the equivalent rectangular masonry stress block and associated ultimate strain capacity for calculation of nominal strength. These parameters do not provide sufficient information for establishing the concrete masonry constitutive relationship because only the initial elastic response and the nominal flexural response are captured. In order to select a suitable constitutive relationship, a literature review on concrete masonry stress-strain behaviour was conducted. Amongst the various research efforts in the past decades, the following two contributions stand out:

A study by Drysdale and Hamid [3-14] considered the behaviour of concrete block masonry under axial compression and provides a good description of the masonry compression failure mechanism in the context of allowable stress design. However, it does not offer essential information on critical strain parameters necessary for ultimate strength calculations, which are essential for capturing the wall ultimate response, such as ultimate strain capacity and strength degradation characteristics.

A concise formulation of the stress-strain relationship for concrete masonry was established by Priestley and Elder [3-2] in the context of strength design. This relationship, covering both unconfined and confined grouted concrete masonry, was based on prism testing and the Kent-Park stress-strain curve for concrete [3-15]. The Priestley-Elder formulation is considered suitable for the present PCM wall study because it describes concrete masonry behaviour under all loading conditions and is based on laboratory testing utilising New Zealand materials. Addi-
tionally, the Priestley-Elder formulation reflects the stress-strain relationship for both low and high strain rates. Fig. 3.6 shows the experimental and theoretical stress-strain curves for unconfined and confined masonry, derived by Priestley and Elder.

3.6.1 Unconfined concrete masonry

Unconfined concrete masonry stress-strain behaviour is shown in Fig. 3.6. It is seen that the experimental relationship (solid line) is nearly linear elastic to a stress of $0.5f_m'$, followed by softening before the maximum strength, $f_m' = 26.5$ MPa, is reached at a strain of approximately 0.0015. Subsequently, rapid strength degradation to approximately $0.2f_m'$ at a strain of 0.004 is observed.

3.6.1.1 Failure mode

Prisms without confining plates failed by separation of the masonry block face shell, resulting in rapid strength degradation [3-2]. Vertical splitting initiated shortly before reaching maximum strength, prior to separation of the face shells. This failure mechanism was attributed to incompatibility of the material properties of the concrete units, grout and mortar. The generally lower strength of the mortar relative to the masonry units and grout, causes the mortar in the
bed joints to expand laterally. Lateral expansion of the mortar towards the centre of the prism is inhibited by the grout, resulting in a net displacement of the mortar towards the outside of the prism, thus pulling the masonry face shells away from the grout core and instigating premature failure.

Priestley and Elder concluded that vertical reinforcement included in the grouted cells did not significantly affect the experimental behaviour.

### 3.6.1.2 Theoretical stress-strain curve

The theoretical stress-strain curve for unconfined masonry (dashed line in Fig. 3.6) is defined by Eqns. 3.10 and 3.11, using the notation adopted in this document. A parabola governs the loading branch up to the maximum strength $f'_m$ at a strain of $\varepsilon_m = 0.0015$. Beyond maximum strength, the masonry strength decreases linearly to a plateau of $0.2f'_m$ at a strain of $\varepsilon_{mp}$, as dictated by the slope $Z_m$.

$$f_m(\varepsilon) = 1.067f'_m \left[ \frac{2\varepsilon}{0.002} - \left( \frac{\varepsilon}{0.002} \right)^2 \right], \quad \varepsilon < \varepsilon_m$$

$$f_m(\varepsilon) = f'_m [1 - Z_m(\varepsilon - 0.0015)], \quad \varepsilon_m \leq \varepsilon \leq \varepsilon_{mp}$$

$$f_m(\varepsilon) = 0.2f'_m, \quad \varepsilon > \varepsilon_{mp}$$

(3.10)

The slope of the descending branch is given by:

$$Z_m = \frac{0.5}{\frac{3 + 0.29f'_m}{145f'_m - 1000} - 0.002} \quad \text{and} \quad \varepsilon_{mp} = \frac{0.8}{Z_m} + \varepsilon_m$$

(3.11)

Eqn. 3.10 directly implies an initial elastic modulus ($\varepsilon = 0$) of $E_m$ and an average elastic modulus for the stress range $0-0.33f'_m$ of $\bar{E}_m$:

$$E_m = 1067f'_m$$

(3.12)

$$\bar{E}_m = 976f'_m$$

(3.13)

Fig. 3.7(a) schematically defines the theoretical stress-strain curve parameters for unconfined masonry.
3.6.2 Confined concrete masonry

Confinement of concrete members is usually achieved by means of closely spaced steel reinforcement hoops (spiral or square) [3-15]. In a retrofit situation, confinement can be achieved by external steel or fibre (glass or carbon) jacketing. Neither of these options appear to be appropriate for confinement of concrete masonry.

It is difficult to incorporate reinforcement hoops in ordinary concrete block masonry construction because of the modular geometry. The hoops would necessarily have to be placed in the bed joints, where it is difficult to find the required space considering a typical joint thickness of 10 mm. Placing steel reinforcement hoops in the grout core would result in inefficient confinement because confinement of the grout core would not restrain the masonry block face shells and prevent premature failure by spalling of the face shells.

It was decided to revisit the use of confining plates of the type shown in Fig. 3.4. This technique was developed for concrete masonry in the early 1980s [3-2]. The technique has not been widely used, mainly due to the increased complexity of construction. Ductile reinforced concrete masonry walls subjected to in-plane loading generally develop a large plastic hinge zone with a vertical extent of the order of 50% of the wall length. Confining plates are therefore potentially required over the entire first storey. Fortunately, a much smaller confining zone is required for PCM walls because the unbonded tendons and associated rocking response tend to concentrate plastic deformation in a short zone just above the wall base, with a length of the order of 1 meter. It follows that the shorter plastic zone results in higher strain demands imposed on the lower corners of an unbonded PCM wall.
As stated above, efficient confinement of concrete masonry can be achieved by embedding of steel plates in the horizontal bed joints during construction as shown in Fig. 3.4. As a result of the confining effect of the embedded steel plates, the concrete masonry behaves in a ductile manner as shown in Fig. 3.6 [3-2]. It is seen from the figure that the measured stress-strain curve (solid line) is nearly linear elastic in the 0-0.5f'_m stress range, followed by softening before reaching the maximum strength of 32 MPa at a strain of approximately 0.0020. Subsequently gradual strength degradation to approximately 0.2f'_m at a strain of approximately 0.012 is observed. It is noted, that the prisms used to established the experimental curves for both the unconfined and confined masonry were constructed at the same time of blocks and grout from the same batches. Comparing the strength of the confined prisms with that of the unconfined prisms, a strength enhancement of approximately 1.2 is found.

3.6.2.1 Failure mode

Including confining plates in the bed joints changed the prism failure mode. Vertical splitting was virtually eliminated due to constraint of the mortar by the confining plates. Ductile failure resulted from gradual strength degradation associated with a shear-compression type of failure, generally occurring within one masonry course.

3.6.2.2 Theoretical stress-strain curve

The theoretical stress-strain curve (dashed line in Fig. 3.6) is defined by Eqns. 3.14. A parabola governs the loading curve up to the maximum strength 1.067Kf'_m at a strain of \( \varepsilon_m = 0.002K \). Beyond maximum strength, the masonry strength decreases linearly to a plateau of \( 0.2 \times 1.067Kf'_m \) at a strain of \( \varepsilon_{mp} \), as dictated by the slope \( Z_m \).

\[
\begin{align*}
    f_m(\varepsilon) &= 1.067Kf'_m \left[ \frac{2\varepsilon}{0.002K} - \left( \frac{\varepsilon}{0.002K} \right)^2 \right], \quad \varepsilon < \varepsilon_m \\
    f_m(\varepsilon) &= 1.067Kf'_m \left[ 1 - Z_m(\varepsilon - 0.002K) \right], \quad \varepsilon_m \leq \varepsilon \leq \varepsilon_{mp} \\
    f_m(\varepsilon) &= 0.2 \times 1.067Kf'_m, \quad \varepsilon > \varepsilon_{mp}
\end{align*}
\]

(3.14)

where \( f'_m \) is the unconfined strength related to testing of unconfined prisms. This equation is analogous to Eqn. 3.10, except for the factor K that accounts for the prism strength increase and strain increase due to the confining plates.
\[ K = 1 + \rho_s \frac{f_{yh}}{f_m'} \]  

(3.15)

where \( \rho_s \) is the volumetric confining ratio and \( f_{yh} \) is the yield strength of the confining steel. Normally the transverse confining ratios, \( \rho_u \), for confined masonry in the two orthogonal directions are different due to the masonry unit dimensions. Priestley and Elder therefore proposed to use:

\[ \rho_s = 2\rho_a = \frac{2A_p}{h''S_h} \]  

(3.16)

where \( \rho_a \) is the minimum ratio of the two directions and \( A_p \) is the area of the confining plate cut by a vertical section of \( h'' \times S_h \) (length by spacing, respectively). The slope of the descending branch is given by:

\[ Z_m = \frac{0.5}{\frac{3 + 0.29f_{m'}}{145f_{m'} - 1000} + \frac{3}{4} \rho_s \frac{h''}{S_h} - 0.002K} \]

and \[ \varepsilon_{mp} = \frac{0.8}{Z_m} + \varepsilon_m \]  

(3.17)

In the Priestley-Elder study it was calculated from Eqn. 3.16 that \( \rho_s = 0.00766 \) where \( h'' = 390 \) mm (long direction of prism), \( S_h = 200 \) mm and \( f_{hy} = 316 \) MPa were used. In Eqn. 3.17, \( Z_m \) was based on the values defined above, except \( h'' \) was taken as the short dimension of the prism, 140 mm.

Eqn. 3.14 directly implies an initial elastic modulus (\( \varepsilon = 0 \)) of \( E_m \) and an average elastic modulus for the stress range 0-0.33\( \times 1.067Kf_m' \) of \( \bar{E}_m \):

\[ E_m = 1067f_m' \]  

(3.18)

\[ \bar{E}_m = 1021f_m' \]  

(3.19)

where \( f_m' \) is the unconfined prism strength. Fig. 3.7(b) schematically defines the theoretical stress-strain curve parameters for confined masonry.

### 3.6.3 Evaluation of stress-strain relationship

A series of nine masonry prisms were tested as described in section 3.2.3. Prism and confining plate dimensions are shown in Fig. 3.4 and the testing results are shown in Table 3.3. Continuous stress-strain curves were measured in all of the tests. The strain measurements were reliable for strains up to \( \varepsilon_m \), corresponding to the maximum strength. Beyond this point, the failure
pattern did in some cases influence the measurement readings. The explosive failure mode of the unconfined prisms resulted from rapid release of elastic strain energy stored in the testing machine, that could not be resisted by the prism. Ductile behaviour of the confined prisms was experienced, i.e. the prism were able to absorb the elastic strain energy stored in the testing machine, that was released after reaching prism maximum strength.

Fig. 3.8 illustrates the typical failure modes for the unconfined and confined prisms tested. The failures resemble those described above [3-2]: U200 exhibited splitting failure and CP200 exhibited shear/compression failure that occurred within one masonry course. The highly confined prisms, CP100, failed as a result of weld fracture of the confining plates, allowing for lateral expansion of the masonry and initiation of shear/compression failure. Failure for the CP100 prisms extended over one or two half height courses.

It is noted that the prism tests conducted by the author had a height of 600 mm (140 mm thick) and that those tested by Priestley and Elder had a height of 1000 mm (190 mm thick). The shorter prism height used by the author may have caused some degree of confinement due to end platten restraint with the expectation of some augmentation of maximum strength and
associated strain in comparison with those of 1000 mm high prisms of similar material. This may also affect the unloading branch of the prism stress-strain curve with the expectation of increased $e_{mp}$. Comparison with the Priestley and Elder research should therefore be done with the above observation in mind.

### 3.6.3.1 U200, Unconfined prisms

Fig. 3.9 shows a typical measured stress-strain curve for unconfined masonry prisms. Only the rising branch of the curve was measured reliably because of explosive prism failure. A maximum strength of 17.8 MPa was measured at a strain of 0.0021. The theoretical stress-strain curve is also plotted in Fig. 3.9 and appears to describe the rising branch of the stress-strain curve reasonably accurately. The greatest deviation is seen at maximum strength, where the strain theoretically attains 0.0015 while the experimental strain amounts to 0.0021. This discrepancy may be caused by platter restraint for 600 mm high unconfined prisms resulting in some confinement. The theoretical curve predicts that the falling curve reaches the residual plateau of $0.2f_m'$ at $e_{mp} = 0.0066$. The experimental value for $e_{mp}$ of about 0.0052 is associated with a high degree of uncertainty and cannot be relied on conclusively. As noted above, confinement effects from the platter restraint may have increased the measured maximum strength.
In the Priestley-Elder study an average compressive strength of 26.5 MPa was found, and was used for developing the above formulae. Fig. 3.10, showing $\varepsilon_{mp}$ predicted by Eqn. 3.11 as a function of $f'_m$, reveals drastic increase of $\varepsilon_{mp}$ for lower masonry strength when compared to the value associated with $f'_m = 26.5$ MPa. It is the opinion of the author that Eqn. 3.11 exaggerates the masonry strain capacity for unconfined concrete masonry of strength of less than 25 MPa. For example it is found that $\varepsilon_{mp} = 0.0083$ for $f'_m = 16$ MPa, a strength typically encountered for NZ North Island concrete masonry. The explanation for prediction of such high values of $\varepsilon_{mp}$ can be found in the origin of the equation. The Kent-Park curve was determined from concrete research which rarely deals with specified strengths of less than 30 MPa. The equation, having been calibrated for compressive strengths higher than 30 MPa, therefore only serves as an extrapolation for compressive strength below 30 MPa. Consequently, it is proposed to limit $\varepsilon_{mp}$ to 0.0040 (approximately the value predicted for $f'_m = 26.5$ MPa) for unconfined masonry in this document, a realistic value for masonry compressive strength below 26.5 MPa.

Both Eqn. 3.9 and the prism testing results reported in Table 3.3 support the elastic modulus given in Eqn. 3.13, although a discrepancy of about 20% is found between Eqns. 3.8 and 3.13. It is mentioned that the $0.2f'_m$ plateau may not be appropriate for unconfined masonry. Both these deficiencies are, however, expected to not affect prediction of flexural behaviour significantly.

With due consideration to the limited amount of stress-strain data available from prism testing of North Island concrete masonry using pumice aggregate, the Priestley-Elder theoretical stress-strain relation is found acceptable for detailed analysis of unconfined PCM.

### 3.6.3.2 CP200, Confined prisms

Fig. 3.11 shows a typical measured stress-strain curve for confined masonry, using the confining plates shown in Fig. 3.4 ($\rho_s = 0.00577$, $f_yh = 240$ MPa). Stress and strain measurements for the loading part of the curve are associated with little uncertainty. The accuracy of the unloading curve was, however, uncertain because masonry crushing and associated strength degradation occurred over approximately one block height, corresponding to approximately 0.25 to 0.5 times the gauge length of the strain measurement. The post-$\varepsilon_m$ strain measurements shown in Fig. 3.11 were corrected according to the observed location and extent of failure.
Fig. 3.10—Variation of masonry strain $\varepsilon_{mp}$ with masonry strength

![Graph showing the variation of masonry strain with masonry strength.]

Fig. 3.11—CP200, Confined masonry stress-strain curve

![Graph showing the stress-strain relationship for CP200 masonry, including experimental and theoretical predictions.]

Fig. 3.11 also shows the theoretical stress-strain relationship predicted by the Priestley-Elder equations using $f'_m = 17.8$ MPa determined from the U200 prisms. As suggested by the figure, the predicted strength enhancement due to the confining plates was not achieved experimentally. This observation can to some degree be explained by using $f'_m$ from the U200 prisms. It
was stated above that the testing machine plattern may have provided a confining effect to the U200 prisms. In that case the measured U200 strength was superficially increased relative to the true unconfined strength. With that in mind it cannot be excluded that CP200 confining does result in strength enhancement.

In terms of the unloading branch, the experimental curve follows the theoretical curve reasonably closely within the margin of error of the strain measurements. For better approximation of the experimental result, the theoretical Priestley-Elder stress-strain curve was modified by using $K f_m' = f_m'$ in Eqn. 3.14 ($K$ retained in other equations). The resulting curve is shown in Fig. 3.11 labelled ‘Priestley-Elder Modified’ and reveals a better fit with the average experimental curve.

The initial elastic modulus given by Eqn. 3.19 appears to be approximately 17% higher than the value found by experiment and listed in Table 3.3. Note that $f_m'$ in Table 3.3 denotes the maximum strength measured while $f_m'$ in Eqn. 3.19 is the unconfined prism strength.

### 3.6.3.3 CP100, Confined prisms

Further confinement of the concrete masonry was achieved by using half height masonry blocks with confining plates spaced vertically at 100 mm, as outlined in Fig. 3.4 ($\rho_s = 0.01154$, $f_{y_h} = 240$ MPa). The experimental and theoretical curves for CP100, shown in Fig. 3.12, exhibit trends similar to those of CP200: exaggerated theoretical strength enhancement but reasonably good correlation between the falling branches of the experimental and theoretical curves. Assuming little strength increase, by taking $K f_m' = f_m'$ in Eqn. 3.14, resulted in generation of the curve labelled ‘Priestley-Elder Modified’. It is seen that very good correlation between the experimental and modified theoretical curves was obtained. It is also seen from Fig. 3.12 that the predicted initial elastic modulus corresponds well with the theoretical curves, particularly for the ‘Priestley-Elder Modified’ curve.

It is observed, given the confining plates used in the experiment, that the strength enhancement predicted by Eqn. 3.14 was exaggerated by a factor of approximately $K$, but that the strain capacity, $\varepsilon_{pm}$, given by Eqn. 3.17 was in agreement with the experimental results. It is again pointed out that $f_m'$ from U200 prisms may have been superficially enhanced due to restraint by the testing machine plattern, making comparison with the Priestley and Elder theory less conclusive.
3.6.4 Effect of strain rate

The dynamic nature of seismic loading results in masonry strain rates in the order of 0.005 strain/sec [3-15]. The general trend is that a high strain rate results in higher masonry peak strength and lower strain capacity. Priestley and Elder [3-2] determined an average strength increase of 17% for a strain rate increase from 0.000005 strain/sec to 0.005 strain/sec for unconfined as well as confined masonry. Experimental stress-strain curves for concrete masonry are normally obtained at low strain rates.

For unconfined masonry, it follows that the theoretical peak strength at high strain rates, to be used in Eqn. 3.10, should be taken as 1.17 times $f_m'$. Similarly, $Z_m$ in Eqn 3.11 should be increased 17%. For confined masonry, $K$ and $Z_m$ in Eqns. 3.14 and 3.17 should be replaced with:

$$K_d = 1.17 \left[1 + \rho_s \frac{f_{yh}}{f_m'}\right]$$

(3.20)

$$Z_{md} = \frac{1.17 \times 0.5}{\frac{3 + 0.29 f_m'}{145f_m' - 1000} + \frac{3}{4} \rho_s \frac{h'}{S_h} - 0.002 K_d}$$

(3.21)
It is again emphasised that $f_m'$ is related to unconfined masonry prism testing at low strain rates.

3.6.5 Tensile strength

The tension strength of grouted concrete masonry is generally negligible in comparison to the compressive strength, is associated with high uncertainty, and normally exerts little influence on wall design in the context of limit state design. The tensile strength is therefore ignored by most limit state design codes, e.g. NZS 4230:1990. Estimates for tensile capacity of grouted concrete masonry subjected to in-plane flexural loading can be found in codes that allow design to elastic theory (allowable stress design), e.g. MJSC [3-8], that defines the tensile capacity (modulus of rupture) as $f_t = 1.72$ MPa. A tensile capacity of the order of 1-2 MPa appears to be appropriate for design.

3.6.6 Unloading properties

The unloading properties describe the unloading path following development of strains larger than $\varepsilon_m$, corresponding to the peak strength, $f_m'$. These properties may influence dynamic analysis of PCM walls, which may be subjected to load reversal many times during a seismic event. There is little information available on unloading properties specific to concrete masonry. The reader is therefore referred to section 9.1.2 for a brief discussion of unloading properties adopted in relation with FEM analysis of PCM walls.

3.7 PRESTRESSING STEEL

High strength prestressing (HSP) steel is the preferred material for prestressing of concrete masonry walls. HSP steel has considerably higher elastic strain capacity than low strength prestressing steel, simply because HSP steel possesses a much higher yield stress than low strength steel, while having similar elastic modulus. The higher strain capacity allows for large elastic tendon elongation and makes the HSP steel prestress less sensitive to losses due to creep and shrinkage of the concrete masonry.

Steel is not the only material option for prestressing of walls. Prestressing of masonry walls using carbon fibre tendons was investigated by e.g. Sayed-Ahmed et al. [3-16] and Holden [3-17]. The investigation by Sayed-Ahmed et al. featured testing of a prestressed clay masonry wall and proved that carbon fibre tendons are a viable option for non-seismic design where little displacement capacity is required. Holden investigated a prestressed concrete wall and sug-
gested that carbon fibre tendons are a viable option for seismic design when the carbon fibre tendons are designed to remain elastic for the required displacement demand. Carbon fibre tendons behave in a linear elastic manner in the entire loading range until maximum strength is attained, followed immediately by rupture. The material is inherently of brittle nature and gives no warning before failure. HSP steel, on the other hand, is ductile and allows for plastic strain far beyond nominal yield strain.

While emphasis in this document is put on tendons remaining elastic during maximum credible ground shaking (design earthquake with approx. 500 year return period), tendon yielding can potentially occur in the event of an extraordinarily strong earthquake (return period of more than 500 years), thus the ability for tendons to deform plastically (yielding) will prevent sudden tendon failure, entail energy dissipation and therefore further protection of the structure. In addition, the focus of this thesis is on application of commonly used and commercially available materials. Therefore, only the use of HSP steel is considered for design of unbonded PCM walls in this document.

3.7.1 Types of high strength prestressing steel

Prestressing steel basically comes in two forms: as flexible strand and as rigid bars. In general terms, flexible strands are appropriate for undulated prestressing profiles and long distances, and for high loads, as they typically have higher yield stress, whereas bars primarily are useful for straight prestressing over shorter distances. Installation of strand solutions is more complex than installation of bar solutions, because strands are anchored individually with wedges or the equivalent and require specialised hydraulic jacks for stressing. Installation of bar solutions, consisting of threaded bars, anchor plates and nuts, requires stressing the bars with a hydraulic jack and tightening of the nut. Alternatively, the bars can be stressed by tightening the nut with a torque wrench or by using calibrated washers and a wrench.

3.7.2 Material properties

The material properties of strands and bars vary considerably, as suggested in Table 3.6, which presents typical material properties [3-18]. Fig 3.13 shows a typical stress-strain curve for high strength prestressing steel. It is noted that there is no yield plateau as in the case for mild reinforcement. The curve is approximately linear elastic up to the yield strength $f_{py}$, then the curve bends over, gradually levelling out, until failure at the stress of $f_{pu}$ and corresponding strain of $\varepsilon_{pu}$. The yield strength $f_{py}$, provided by the manufacturers, is defined as the steel stress corre-
sponding to a non-proportional strain of 0.1%, i.e. the intersection of the loading curve with a line offset of 0.1% strain (or 0.2% depending on applicable testing standard and material) and a slope of $E_{ps}$, refer to Fig. 3.13. This stress is also called the 0.1% proof strength. Prior to calculating $f_{py}$, the elastic modulus $E_{ps}$ is found as the slope of the linear part of the loading curve.

**TABLE 3.6**—Material properties of prestressing steel

<table>
<thead>
<tr>
<th>Type</th>
<th>$f_{py}$</th>
<th>$f_{pu}$</th>
<th>$\varepsilon_{pu}$</th>
<th>$E_{ps}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strand $\phi$15mm</td>
<td>1480</td>
<td>1750</td>
<td>3.5%</td>
<td>180-205</td>
</tr>
<tr>
<td>Bar $\phi$23mm</td>
<td>930</td>
<td>1080</td>
<td>6%</td>
<td>170</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Type</th>
<th>$f_{pu}$</th>
<th>$E_{ps}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>MPa</td>
<td>MPa</td>
<td>GPa</td>
</tr>
</tbody>
</table>

**3.8 CONCLUSIONS**

It was found that accurate prediction of $f'_m$ and $\varepsilon_{mu}$ is necessary for design of PCM walls because of the significant influence of these parameters on the wall strength and displacement capacity. NZS 4230:1990 needs to move towards a system for specification of $f'_m$ based on the characteristic strength of the blocks and grout, because the current grade dependent approach poorly reflects actual masonry strength. In fact, a characteristic strength $f'_m$ of 12.8 MPa can safely be specified by designers in New Zealand without the need for verification through material testing. Prism testing should, where possible, be employed to exploit the available strength fully, which often exceeds 16 MPa for standard products.

Using confinement plates in the bed joints is an efficient way of improving the masonry strain capacity. Uniaxial strain capacity of beyond 1% can readily be achieved. The ultimate masonry

![Prestressing steel stress-strain relationship](image)

**Fig. 3.13**—Prestressing steel stress-strain relationship
strain, $\varepsilon_{mu}$, should be related to the PCM wall flexural behaviour because the expected maximum wall strength is associated with an extreme masonry fibre strain far larger than the codified limits. Appropriate equivalent rectangular stress block parameters need to be established for PCM walls, with due consideration to confinement.

It is recommended to use Eqn. 3.8 for prediction of $E_m$ for New Zealand conditions based on the likely masonry crushing strength, and Eqn. 3.9 when using characteristic strength.

Theoretical stress-strain curves may be established using the Priestley-Elder formulation. The only parameters required to define the curves are the masonry geometry, the expected unconfined strength, $f'_m$, and if present the confining plate dimensions and yield strength. For unconfined masonry, it appears that $\varepsilon_{mp}$ is overpredicted for masonry strengths below 25 MPa, thus $\varepsilon_{mp}$ should be limited to 0.004. For low volumetric confining steel ratios, $\rho_s$, the Priestley-Elder formulation was found to over-predict the strength enhancement, however, the predicted strain capacity appears reasonable. The strength enhancement $Kf'_m$ given in Eqn. 3.14 could conservatively be taken as just $f'_m$.

The tensile strength of grouted concrete masonry is unreliable and should be neglected for limit state design.

It was concluded that it is essential to use high strength steel for prestressing of PCM walls because of the beneficial effect of the large elastic elongation capacity and reduction of the time dependent losses.
3.9 REFERENCES


Chapter 4

TIME DEPENDENT EFFECTS

4. ASSESSMENT OF PRESTRESSING LOSSES

Assessment of prestressing losses is of great importance for prestressed concrete masonry structures. The effective prestress level decreases over time due to creep, shrinkage and steel relaxation, hence reducing structural efficiency. Research has indicated that prestress losses in prestressed concrete masonry may range as high as 25% of the initial stress level, depending on the masonry, the initial stress level and the prestressing hardware [4-1].

Losses are typically attributed to the following factors:

1. Shrinkage
2. Creep
3. Relaxation of tendons
4. Elastic shortening
5. Anchorage ‘draw-in’ (seating losses)
6. Friction due to undulation of tendons
7. Thermal effects

Elastic shortening (4) occurs when a tendon is stressed due to the masonry elastic properties. When more than one tendon is used in a wall and these are stressed one at the time, stressing of any tendon but the first one will theoretically affect the tendon stress in all previously stressed tendons. Elastic shortening can readily be estimated using elastic analysis, but is expected to have minimal influence for PCM because prestressing area ratios typically are relatively low. As a consequence, (4) is only discussed in this chapter as a benchmark for shortening in relation with creep strain. As this chapter primarily is concerned with post-tensioned concrete masonry (PCM) walls with unbonded straight tendons, item (6) is a non-issue. Anchorage draw-in (5) may be estimated according to the utilized prestressing hardware and will not be
discussed further. Short term thermal effects (7) are disregarded in this study because of their reversible nature. Nevertheless, tendon stress change may arise from temperature difference between the masonry and the prestressing steel, which may readily be evaluated. It is noted that the issues 4 through 7 have little or no time dependency. Consequently, this chapter solely discusses the time dependent issues affecting prestress losses in concrete masonry: shrinkage, creep, and prestressing steel relaxation.

Equations for evaluating creep and shrinkage are presented, complying with current practice for design of prestressed concrete and the masonry code provisions from the British standard BS 5628 [4-2], the Australian standard AS 3700 [4-3] and the North American code MJSC [4-4]. A creep and shrinkage experimental program conducted at the University of Auckland is thereafter presented and compared with creep and shrinkage code provisions. Equations for evaluating steel relaxation are presented, followed by current code recommendations. Lastly recommendations are made for calculation of prestress losses under New Zealand conditions.

4.1 THEORETICAL CONSIDERATIONS

4.1.1 Creep and shrinkage of concrete

The following is a short description of the creep and shrinkage mechanisms in concrete (water based) as reported by Neville [4-5]. It is emphasised that the description serves only to provide sufficient understanding of the creep and shrinkage mechanism for development of relevant equations presented below.

Despite the fact that creep and shrinkage are interactive processes and cannot be completely dissociated, the effects of creep and shrinkage traditionally are dissociated for simplicity as illustrated in Fig. 4.1. This is of course an extremely simplified description of a very complex interaction between time dependent effects. The following shortening of a concrete specimen is observed when subjected to sustained axial load:

Initial elastic shortening:

The initial elastic shortening, denoted ‘5’ in Fig. 4.1, results from application of axial load at time t₀ and is often used as reference for describing the magnitude of creep and shrinkage. The initial elastic shortening reflects the stress level applied to the concrete and the initial elastic modulus of the concrete.
Shrinkage:

Shrinkage of drying concrete, denoted '4' in Fig. 4.1, is characterised by volume change in the cement paste and can be described by two phenomena: (i) drying shrinkage and (ii) shrinkage due to carbonation.

(i) Drying shrinkage occurs when water is removed from the cement paste because of surrounding unsaturated air. Only water lost from the cement paste causes drying shrinkage. Free water evaporating from cavities, etc., induces little or no shrinkage. Many factors affect drying shrinkage. The physical dimensions, notably the surface to volume ratio, and the concrete aggregate are highly influential.

Storage conditions play an important role. Neville [4-5] states that the relative humidity of the medium surrounding the concrete greatly influences the magnitude of drying shrinkage, thus the drying shrinkage strain of concrete stored in 50% relative humidity can amount to about 150% of the drying shrinkage strain encountered at 70% relative humidity.

The issue of reversibility of shrinkage due to moisture movement is not considered here because it is assumed that PCM walls will not be subjected to drastic change of relative humidity. This effect is only distinct if concrete, which has been allowed to dry in air with a given relative humidity, is subsequently placed in water (or subjected to nearly 100% humidity).

(ii) Shrinkage due to carbonation is caused by a chemical process where carbonic acid reacts with CO₂. The rate of carbonation depends on the moisture content of the concrete, the relative
humidity of the ambient medium and the size of the specimen. Carbonation increases the shrinkage at intermediate relative humidities (RH), but not at high RH (100%) or RH below 35%, either because of pores full of water prohibiting diffusion of CO₂ or because of insufficient water to sustain the chemical process.

Creep:

Creep is defined as gradual increase in strain with time under sustained load. According to Neville [4-5], the nature of creep is still controversial, but is generally attributed to internal pressure caused by external loads in absorbed and interlayer water within the microstructure of the cement paste. This pressure causes migration of the water and resulting volumetric change. In reality, it is the cement paste that undergoes creep. The aggregate undergoes little creep and only affects the overall creep through its function, primarily being that of restraint to the cement paste. Fig. 4.1 schematically illustrates the change in strain of an axially loaded and drying concrete specimen as a function of time. It is seen that the total creep is divided into two components: (i) basic creep and (ii) drying creep.

(i) The basic creep, denoted ‘2’ in Fig. 4.1, is related to the moisture content in the concrete paste that is available for evaporation (no moisture movement to or from the ambient medium). A reduction of the available moisture content reduces the propensity for viscoelastic movement and therefore results in lower basic creep.

(ii) The second term, drying creep denoted ‘1’ in Fig. 4.1, occurs when an axially loaded specimen is not in hygral equilibrium with its surroundings, meaning that moisture is transferred between the specimen and the surrounding air; thus drying while under load increases the creep of concrete. As concrete dries, the rate of creep reduces and approaches zero. One of the most important external factors influencing creep is the relative humidity. The lower the humidity, the larger the creep rate. At this stage both basic and drying creep occurs. At a later age when the concrete has reached hygral equilibrium with the surrounding medium, the creep rate stabilises, and remains the same irrespective of relative humidity level. The latter creep rate is attributed to basic creep because of hygral equilibrium. It appears that the rate of creep is little affected by ambient temperature when the fluctuations about room temperature (20 °C) are small. However, the temperature directly affects relative humidity.

Alternating wetting and drying may result in an increase of the magnitude of creep. This phenomena, termed ‘wetting creep’, suggests that results of laboratory tests may underestimate the
creep under normal weather conditions [4-5]. Bryant et al. [4-6] came to the opposite conclusion from comparison of creep strains in concrete specimens placed (1) on top of a bridge deck (alternating wetting and drying) and (2) inside the box girder of the same bridge (sheltered environment). Bryant et al. found that creep strain after 600 days for (1) amounted to about 90% of the creep strain of (2).

Long term creep and shrinkage:

Fig. 4.1 illustrates that at a certain age, $t$, nearly all creep and shrinkage have occurred. According to Neville [4-5] creep and shrinkage take place over long periods. Typically, the shrinkage after 1 year amounts to about 75% of the shrinkage after 20 years. If the shrinkage strain at 1 year is taken as unity, the shrinkage strain after 20 years amounts to $4/3$. Similar observation is made for concrete creep, where the creep strain after 20 years may amount to $4/3$ of the creep strain after 1 year. This has significant influence on the estimate of prestress loss in a structure, when a service life of more than 20 years commonly is considered, given that most creep and shrinkage experiments last 1 - 2 years.

According to Bryant and Vadhanavikkit [4-7], the final amount of creep and shrinkage strain depend strongly on size of the concrete member. Thicker members, i.e. members with further distance between the concrete core and the exterior surfaces, show slower creep and shrinkage strain rates and lower long term creep and shrinkage strain, than thin members subjected to similar stress level and environment. Modelling of the strain rate and long term strain dependencies of member dimensions can for example be found in the CEB-FIP model code [4-8] and Eurocode 2 [4-9]. The ACI approach in [4-10] does not take concrete dimension into account in estimation of creep and shrinkage properties.

4.1.2 Creep and shrinkage of concrete masonry

Creep and shrinkage of concrete masonry, a material combination composed of water-based cementitious materials, are expected to eventuate in a fashion similar to those of concrete as described above. However, there are differences between concrete and concrete masonry, such as the higher porosity of concrete masonry, the fact that concrete masonry blocks are precast and normally autoclaved (steam cured and moisture controlled) and the presence of mortar joints. The nature of creep and shrinkage for fully grouted masonry is expected to be similar to that of concrete because of the presence of wet concrete (grout, block fill) in about 50% of the cross section after grouting.
It appears that drying shrinkage in concrete masonry can be reversed to a considerable extent because of the inherent porosity of the concrete masonry blocks, allowing water to easily penetrate deep within the specimen. This can be prevented with a suitable water repellent coating.

According to Lenczner [4-11], who studied clay masonry, about 95% of all creep can be expected to occur within 1 year. This information is not confirmed to apply for concrete masonry. As discussed above for concrete, perhaps 75% of the total creep and shrinkage (after more than 20 years) for concrete occurs within 1 year (strongly dependent on member thickness). For concrete masonry a larger proportion of the long term shortening is expected to occur within one year because of thin dimensions and high porosity which lead to hygral equilibrium with the surroundings at an early stage. This is in particular the case for the concrete masonry shrinkage. Consequently, nearly all shortening due to both creep and shrinkage takes place within 5 years of construction and load application. This statement does not hold true if concrete masonry is subjected to extreme humidity variations which, as noted above, may reverse the shrinkage process and may instigate wetting creep.

It is assumed that creep and shrinkage of concrete masonry can be identified as independent and additive effects, as outlined in Fig. 4.1; a simplification of a very complex interaction between the time dependent effects. It appears, however, that more complex theoretical consideration is little warranted in the context of concrete masonry because of significant scatter of creep and shrinkage results from laboratory testing [4-12], and because of wide variation of material properties for concrete blocks, mortar and grout.

4.1.3 Mathematical expressions

This section considers the total prestressing loss due to creep, shrinkage and prestressing steel relaxation, meaning the long term loss after say 20 years, when nearly all axial movement in the concrete masonry and all relaxation of the prestressing steel have taken place. The initial prestressing force after anchorage lock-off, \( P_I \), and the long term prestressing force, \( P_t \), are important quantities to the structural designer, who must ensure adequate structural capacity at any given time in a structures service life. The variation of the prestressing force with time is not discussed in this section because such detail is not warranted for this study. However, the implications of time variation of shrinkage, creep and relaxation are briefly discussed in section 4.5.2.
4.1.3.1 Creep strain

The total creep strain may be expressed by Eqn. 4.1. It is assumed that the concrete masonry creep strain $\varepsilon_{cr}$ is proportional to the initial elastic deformation $\varepsilon_{mi}$ by the factor $C_c$, called the ‘creep coefficient’.

$$\varepsilon_{cr} = C_c \varepsilon_{mi} = C_c \frac{f_{mi}}{E_m}$$ (4.1)

In this equation, $f_{mi}$ is the initial axial stress on the masonry cross section after anchor lock-off (prestressing + gravity load) and $E_m$ is the initial elastic modulus of masonry defined in Chapter 3. Assuming that $E_m$ remains nearly constant, Eqn. 4.1 can be reduced to:

$$\varepsilon_{cr} = k_c f_{mi}$$ (4.2)

where $k_c$ is termed the ‘specific creep’. Eqn. 4.2 is a greatly simplified expression. While perhaps troublesome to researchers, it has considerable appeal to codification and design work because of it's simplicity. The total prestressing loss due to creep, $\Delta f_{cn}$ is derived from Eqn. 4.1 as:

$$\Delta f_{cr} = \varepsilon_{cr} E_{ps} = \frac{E_{ps}}{E_m} C_c f_{mi}$$ (4.3)

where $E_{ps}$ is the elastic modulus of the prestressing steel defined in Chapter 3. When assuming that $E_m$ remains nearly constant this equation reduces to:

$$\Delta f_{cr} = \varepsilon_{cr} E_{ps} = E_{ps} k_c f_{mi}$$ (4.4)

The material creep parameters $C_c$ and $k_c$ are specific to the implicated materials (concrete block, mortar and grout) and are derived experimentally. In the event of an experimental investigation, the initial elastic shortening strain considered in Eqn. 4.1, $\varepsilon_{mi}$, is evaluated as strain arising immediately from application of axial load.

It is clear that the Eqns. 4.3 and 4.4 are reflecting prestress loss due to creep for walls with uniform stress applied to any cross section. If significant permanent moment is applied to a cross section, the end of the wall with the largest compressive stress will experience more shortening than the end with the lowest compressive stress. Such differential creep may lead to non-uniform prestress loss, i.e. tendons placed in opposite ends of a wall with a permanent moment applied may experience different magnitude of prestress loss. For the case of ordinary rectangular PCM walls, this issue is normally of little concern because of symmetrical tendon distri-
bution in the wall cross section and insignificant permanent wall moment. In fact this issue is mostly of concern in prestressed beams and girders that carry permanent load in flexure. The 'effective modulus' method, e.g. Sritheran and Fenwick [4-13], may be employed to capture creep effects due to combined axial and flexural loading.

4.1.3.2 Shrinkage strain

Eqn. 4.5 defines the prestressing loss due to shrinkage of the masonry, \( \Delta f_{sh} \). It is assumed that the loss is proportional to the total shrinkage strain \( \varepsilon_{sh} \), which is independent of axial load on the masonry.

\[
\Delta f_{sh} = \varepsilon_{sh}E_{ps}
\] (4.5)

The material shrinkage parameter \( \varepsilon_{sh} \) is obtained experimentally.

4.1.3.3 Relaxation of prestressing steel

Relaxation of prestressing tendons (general term for both strand and bar) is a well documented property which may be extracted from the prestressing codes. The steel relaxation properties should preferably be provided by the manufacturer. The following values presented in Table 4.1 may be considered typical of modern prestressing strand and bars [4-14] which normally are of the 'low-relaxation'-type. These values represent minimum requirements to prestressing steel. The relaxation is given as a percentage of the initial reference prestressing steel stress of 70% of the breaking load, \( 0.7f_{pul} \), and is measured after 1000 hours. Long term relaxation loss may conservatively be estimated to 3 times the loss after 1000 hours [4-9].

When it is necessary to evaluate relaxation as a function of time, the Prestressed Concrete Institute [4-15] proposes the following equation for calculating the prestressing loss \( \Delta f_{pr} \) due to relaxation of low-relaxation prestressing strands.

\[
\Delta f_{pr}(t) = f_{psl} \left[ \frac{Log_{10}(t)}{45} \left( \frac{f_{psi}}{f_{py}} - 0.55 \right) \right] \quad \text{for} \quad \frac{f_{psi}}{f_{py}} \geq 0.55
\] (4.6)

<table>
<thead>
<tr>
<th>Type</th>
<th>Relaxation ( k_r )</th>
<th>Time</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strand ( \phi )15mm</td>
<td>2.5%</td>
<td>1000</td>
<td>( 0.7f_{pul} )</td>
</tr>
<tr>
<td>Bar ( \phi )23mm</td>
<td>4.0%</td>
<td>1000</td>
<td>( 0.7f_{pul} )</td>
</tr>
<tr>
<td>% of ( 0.7f_{pul} )</td>
<td>Hours</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
In this equation $f_{ps_i}$ is the initial steel stress after anchor lock-off, $t$ is the time given in hours after stressing and $f_{py}$ is the yield stress for the steel. Eqn. 4.6 only applies to prestressing strand with prestress levels above 0.55$f_{py}$. For application to prestressed structural components with prestressing of less than 0.55$f_{py}$ insignificant relaxation is anticipated. Relaxation after 50 years calculated with Eqn. 4.6 amounts to 3.4%. This is significantly less than the previously discussed and probably conservative estimate of 3 times $k$ (1000 hours) = 7.5%.

The above information may be synthesised by the following equation that evaluates the long term prestressing force loss in strand due to relaxation, $\Delta f_{pr}$, by interpolation between zero loss at 0.55$f_{py}$ and 0.7$k$ $f_{pu}$ at 0.7$f_{pu}$, assuming that $f_{py}/f_{pu} = 0.85$:

$$\Delta f_{pr} = k_r f_{ps_i} \times 3.7 \left( \frac{f_{ps_i}}{f_{py}} - 0.55 \right) \quad \text{for} \quad \frac{f_{ps_i}}{f_{py}} \geq 0.55 \quad (4.7)$$

### 4.1.3.4 Total Prestress Loss

The total long-term prestressing loss due to creep, shrinkage and steel relaxation may be evaluated by:

$$\Delta f_{pl} = \Delta f_{cr} + \Delta f_{sh} + \Delta f_{pr} \quad (4.8)$$

Eqn. 4.8 is of conservative nature because the prestress losses are based on the initial masonry stress $f_m$. In reality, the gradually occurring creep, shrinkage and relaxation gradually reduces the masonry stress which then reduces the creep and relaxation rates. Section 4.5.2 discusses these implications in some detail.

Mild steel reinforcement parallel to the prestressing steel effectively resists the vertical creep and shrinkage strain movement and consequently reduces the prestress loss. This effect is not considered herein because it is assumed that prestressing steel constitutes all flexural reinforcement for PCM walls.

For later discussion of structural performance, it follows that the total long term tendon force, $P_t$, following all losses is given by:

$$P_t = P_i - A_{ps} \Delta f_{pl} \quad (4.9)$$

where $P_i$ represents the total initial prestress force after anchor lock-off and $A_{ps}$ is the total prestressing steel area.
4.2 CREEP AND SHRINKAGE EXPERIMENT

A considerable number of research projects have been conducted in various countries on creep and shrinkage of concrete masonry, assessing the magnitude of creep and shrinkage as a function of materials, axial load, time and ambient temperature and humidity [4-12,4-16,4-17]. Results from these experimental investigations, that all lasted about one year, constitute the basis for codification of creep and shrinkage in BS 5628 [4-2], AS 3700 [4-3] and MSJC [4-4].

The precast hollow core concrete masonry blocks ordinarily used in the upper North Island of New Zealand are composed of a porous pumice type of aggregate. With a mass of approximately 1850 kN/m³, this type of block can be classified as light-weight in the New Zealand context. The MSJC working draft [4-4] classifies concrete masonry in three weight groups: (1) light-weight with mass less than 1680 kg/m³, (2) medium-weight with mass 1680-2000 kg/m³ and (3) normal-weight with mass higher than 2000 kg/m³. The North Island concrete masonry block falls in the medium-weight category according to the North America weight specification. Normal-weight concrete masonry blocks are commonly used elsewhere in New Zealand, for example in the Canterbury region where granite aggregate (or other dense aggregate types) is readily available.

None of the studies previously reported apply directly to the New Zealand light-weight type of concrete masonry and none of the creep and shrinkage experiments directly addressed fully grouted concrete masonry, to the knowledge of the author. It was therefore considered necessary to conduct a creep and shrinkage study on locally manufactured concrete masonry. The University of Auckland study presented herein was initiated in collaboration with researchers at the University of Wyoming U.S.A., who initiated a parallel creep study in 1996 and later reported the findings in [4-12].

4.2.1 Design of the experiments

The experiments described in this chapter were conceived to reflect common concrete masonry construction practice in New Zealand and to reflect realistic axial prestress levels expected in PCM. In the year of 1998 it was decided to manufacture testing equipment for 10 wallettes, 1.0 m high, 0.59 m wide and 0.14 m thick. The wallette dimensions are shown in Fig. 4.2.
All walls in the first testing series were constructed of ordinary grey 15 series precast hollow block concrete masonry manufactured to the specifications of NZS 3102:1983 [4-18]. The wallets were built in running bond. The mortar used was pre-blended Trade Mortar™, a mortar used for nearly all concrete masonry construction in New Zealand, consisting of cement, sand and a plasticiser with a mix ratio cement:sand of 1:3. Trade Mortar™ conforms to NZS 4210:2001 [4-19] with a minimum compressive strength of 12.5 MPa. Five wallets were grouted, one was partially grouted (2 out of 3 flues grouted) and four remained ungrouted. Prestress was applied at day 17 after construction (day 11 after grouting) with levels ranging between 0.97 MPa and 2.8 MPa. The grout was specified according to NZS 4210:2001 with a minimum compressive strength of 17.5 MPa. A shrinkage compensating agent, SIKA CAVEX™, was added to the grout shortly before pouring to compensate for immediate shrinkage caused by water removed from the grout by the dry concrete masonry blocks. Two wallets were not stressed, providing an estimate of pure shrinkage shortening. Shortening of the wallets and ambient temperature were monitored on a regular basis throughout the 373 day test duration (counted from the day of stressing). Specifications for testing series 1 are given in Table 4.2.

The following net areas were used for computation of the initial stress, $f_{mi}$: For ungrouted wallets it was assumed that only the block flanges (30 mm wide) and the ends (30 mm wide) were carrying load, as the intermediate webs alternated location and therefore did not carry
TABLE 4.2—Specifications for series I

<table>
<thead>
<tr>
<th>Wall</th>
<th>Grouting</th>
<th>day 0</th>
<th>day 373</th>
<th>% of $f_{mi}$</th>
<th>Concrete masonry properties</th>
<th>Load ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$f_{mi}$</td>
<td>$f_{mf}$</td>
<td></td>
<td>day 28</td>
<td>day 373</td>
</tr>
<tr>
<td>1-UG1</td>
<td>Ungrounded</td>
<td>0.00</td>
<td>0.00</td>
<td>-</td>
<td>15.0</td>
<td>-</td>
</tr>
<tr>
<td>1-UG2</td>
<td>Ungrounded</td>
<td>1.40</td>
<td>1.32</td>
<td>6%</td>
<td>15.0</td>
<td>-</td>
</tr>
<tr>
<td>1-UG3</td>
<td>Ungrounded</td>
<td>2.10</td>
<td>1.92</td>
<td>8%</td>
<td>15.0</td>
<td>-</td>
</tr>
<tr>
<td>1-UG4</td>
<td>Ungrounded</td>
<td>2.80</td>
<td>2.60</td>
<td>7%</td>
<td>15.0</td>
<td>-</td>
</tr>
<tr>
<td>1-PG1</td>
<td>Partially grouted</td>
<td>2.70</td>
<td>2.22</td>
<td>18%</td>
<td>16.7</td>
<td>-</td>
</tr>
<tr>
<td>1-FG1</td>
<td>Fully grouted</td>
<td>0.00</td>
<td>0.00</td>
<td>-</td>
<td>17.5</td>
<td>20.1</td>
</tr>
<tr>
<td>1-FG2</td>
<td>Fully grouted</td>
<td>0.97</td>
<td>0.93</td>
<td>4%</td>
<td>17.5</td>
<td>20.1</td>
</tr>
<tr>
<td>1-FG3</td>
<td>Fully grouted</td>
<td>1.22</td>
<td>1.10</td>
<td>10%</td>
<td>17.5</td>
<td>20.1</td>
</tr>
<tr>
<td>1-FG4</td>
<td>Fully grouted</td>
<td>1.46</td>
<td>1.34</td>
<td>8%</td>
<td>17.5</td>
<td>20.1</td>
</tr>
<tr>
<td>1-FG5</td>
<td>Fully grouted</td>
<td>1.94</td>
<td>1.57</td>
<td>19%</td>
<td>17.5</td>
<td>20.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MPa</td>
<td>MPa</td>
<td>-</td>
<td>MPa</td>
<td>MPa</td>
</tr>
</tbody>
</table>

*estimated, †measured on prisms cut from walltes

load. A resulting net area of 40200 mm² was found. The cross-sectional areas of the fully grouted and partially grouted walltes were taken as 80300 mm² (gross area) and 55100 mm² (2/3 of the gross area), respectively.

A second test series of 10 walltes, using similar materials and construction practice, was initiated in the year of 2000 with the purpose of confirming and expanding the findings from series 1. A wider range of prestress was applied to the walltes, 5 being grouted and 5 ungrouted. The testing setup used for testing of the series 1 walltes was reused and featured improved instrumentation. Prestress was applied at 10 days after construction (9 days after grouting). These walltes were subjected to prestress levels ranging between 1.0 MPa and 4.0 MPa. Two walltes remained unstressed, providing assessments of the pure shrinkage strain. Shortening of the walltes was monitored on a regular basis throughout the 388 day test duration, while ambient temperature and humidity were measured continuously. Specifications for testing series 2 are given in Table 4.3.

Table 4.4 provides schedules of events for both testing series, again with day zero taken as the day of stressing. The age of the blocks could not be obtained accurately but presumably ranged between 28-52 days at the day of construction.

4.2.2 Test setup and instrumentation

Fig. 4.3 shows schematically the test setup and instrumentation configuration for the two testing series. Heavy coil springs were installed in series with the walls between the tendon
TABLE 4.3—Specifications for series 2

<table>
<thead>
<tr>
<th>Wall</th>
<th>Grouting</th>
<th>Prestress $f_{m}$</th>
<th>Concrete masonry properties</th>
<th>Load ratio $f_{m28}/f_{m}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>day 0 $f_{mi}$</td>
<td>day 388 $f_{mf}$</td>
<td>change $%$ of $f_{mi}$</td>
</tr>
<tr>
<td>2-UG1</td>
<td>Ungrounded</td>
<td>0.00</td>
<td>0.00</td>
<td>-</td>
</tr>
<tr>
<td>2-UG2</td>
<td>Ungrounded</td>
<td>1.14</td>
<td>1.16</td>
<td>-2%</td>
</tr>
<tr>
<td>2-UG3</td>
<td>Ungrounded</td>
<td>2.14</td>
<td>2.10</td>
<td>1%</td>
</tr>
<tr>
<td>2-UG4</td>
<td>Ungrounded</td>
<td>2.99</td>
<td>2.96</td>
<td>0%</td>
</tr>
<tr>
<td>2-UG5*</td>
<td>Ungrounded</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2-FG1</td>
<td>Fully grouted</td>
<td>0.00</td>
<td>0.00</td>
<td>-</td>
</tr>
<tr>
<td>2-FG2</td>
<td>Fully grouted</td>
<td>1.00</td>
<td>1.07</td>
<td>-6%</td>
</tr>
<tr>
<td>2-FG3</td>
<td>Fully grouted</td>
<td>1.82</td>
<td>1.67</td>
<td>8%</td>
</tr>
<tr>
<td>2-FG4</td>
<td>Fully grouted</td>
<td>2.78</td>
<td>2.31</td>
<td>17%</td>
</tr>
<tr>
<td>2-FG5</td>
<td>Fully grouted</td>
<td>4.00</td>
<td>3.41</td>
<td>15%</td>
</tr>
</tbody>
</table>

*failed by vertical splitting during stressing sequence, \# estimated, † measured by testing of 3 prisms

anchorages in order to keep the prestressing force approximately constant over time. Both walllette series were built in the Civil Test Hall. Series 1 walllettes were stored in the Civil Test Hall while the series 2 walllettes were relocated to the basement of the Test Hall after application of axial load.

Axial shortening strains were measured via a digital gauge between measurement points. Each measurement sequence (one measurement over each set of measurement points for all walls) that was taken at a particular time was repeated three times to improve the reliability of the data. Test series 1 had the Demec measurement points attached to the 32 mm end-plates in each of the four corners (measurements S1-1 to S1-4), resulting in a gauge length of 1000 mm. The Demec instrumentation layout for testing series 2 was improved so that, for each wallette, 8 measurements were taken directly on the masonry (measurements S2-1 to S2-8) over a gauge

TABLE 4.4—Creep and shrinkage testing, schedule of events

(a) Series 1

<table>
<thead>
<tr>
<th>Day:</th>
<th>Event:</th>
</tr>
</thead>
<tbody>
<tr>
<td>-17</td>
<td>construction</td>
</tr>
<tr>
<td>-11</td>
<td>grouting</td>
</tr>
<tr>
<td>-3</td>
<td>capping</td>
</tr>
<tr>
<td>0</td>
<td>application of axial load</td>
</tr>
<tr>
<td>373</td>
<td>data collection concluded</td>
</tr>
<tr>
<td>400</td>
<td>testing of prisms cut from walllettes</td>
</tr>
</tbody>
</table>

(b) Series 2

<table>
<thead>
<tr>
<th>Day:</th>
<th>Event:</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10</td>
<td>construction</td>
</tr>
<tr>
<td>-9</td>
<td>grouting</td>
</tr>
<tr>
<td>-3</td>
<td>capping</td>
</tr>
<tr>
<td>0</td>
<td>application of axial load</td>
</tr>
<tr>
<td>2</td>
<td>2-UG4 split vertically between two cells</td>
</tr>
<tr>
<td>70</td>
<td>2-UG3 split vertically between two cells</td>
</tr>
<tr>
<td>230</td>
<td>2-FG5 split vertically down the middle</td>
</tr>
<tr>
<td>388</td>
<td>data collection concluded</td>
</tr>
<tr>
<td>388</td>
<td>testing of prisms cut from walllettes</td>
</tr>
</tbody>
</table>
length of 200 mm. Several improvements were anticipated: (i) strain was measured in a uniform stress region, (ii) measurements were not influenced by potential deformation of the end-plates, and (iii) more measurements were obtained with improved reliability.

A reference gauge length of 200 mm was provided by a reference bar made of INVAR steel with a very low coefficient of thermal expansion. This bar was measured 3 times for each measurement sequence for testing series 1 (before wallette 1, after wallette 5 and after wallette 10) and 6 times for each measurement sequence for series 2 (before and after every two wallettes). It was found using the Demec system that length measurements (and changes) could be replicated with good accuracy for repeated measurement sequences taken at the same day.

The following procedure was used to determine the axial shortening strain of a wallette at a particular time, \( t \), having acquired three measurement sequences at time \( t \):

1. For each sequence: verify the reliability of reference measurements (comparison between the three sequences); subtract the average reference measurement from the wallette measurements taken between these two reference measurements. Then verify the reliability of the corrected wallette measurements (comparison between the three sequences).

2. For each data set (three corrected measurement sequences for a particular day), find the average measurement for each wallette (i.e. average of \( 3 \times 4 \) measurements for series 1 and of \( 3 \times 8 \) measurements for series 2).
3. The shortening at time, \( t \), relative to the day of reference (day of stressing), \( t_0 \), is found by subtraction of the wall average shortening at time \( t_0 \) from the average obtained at time \( t \). Finally, the shortening strain is found by division of the shortening at time \( t \) with the measurement gauge length (1000 mm for series 1 and 200 mm for series 2).

4.2.3 Series 1 experimental results

Fig. 4.4 shows the shortening strain (shrinkage + creep) curves for all wallets, where the two curves for the unstressed wallets reflect the pure shrinkage strain. Fig. 4.5 shows creep strain curves for all data, sampled over 373 days. The creep results presented for grouted, partially grouted and un-grouted concrete masonry are not corrected for temperature and humidity variations throughout the sampling period.
Fig. 4.5—Series 1, creep strain

Fig. 4.6 shows the variation of the ambient air temperature measured concurrently with the creep and shrinkage measurements. It is noted that the measurements and the smoothed curve (thick line) reflect the daytime temperature in the Civil Test Hall. The mean 24 hour temperature was probably a few degrees Celsius lower on average.

From Fig. 4.4 it appears that the shortening strain curves stabilised over the last 50 days of measurements, and for the unloaded wallets (pure shrinkage) approximately levelled out. This observation is supported by Fig. 4.6, showing increasing temperature over the last 50 days of the experiment, a condition that generally would have indicated an increase of the shrinkage rate. It would be reasonable to assume that the maximum shrinkage strains for un-grouted concrete masonry amount to approximately 200×10⁻⁶ and for grouted concrete masonry amount to approximately 450×10⁻⁶. The shrinkage strain for the partially grouted wallette was estimated
Fig. 4.6—Series 1, ambient air temperature variation

as 2/3 of the grouted wallette value plus 1/3 of the ungrouted wallette value, reflecting the ratio of filled to unfilled flues in the partially grouted wallette (same assumption was used in Fig. 4.5(b)). Fig. 4.6 shows that the ambient temperature at the beginning and end of the test were similar. This suggests that thermal expansion of the wallettes did not affect the final strain readings because of no temperature difference.

Considering Figs. 4.5(a) and (b), it is evident that the creep strain curves have approximately levelled out by the end of the experiment. In fact, the creep curves for the fully and partially grouted wallettes levelled out as early as 4 months (200 days) after application of axial load. It is therefore assumed that most of the creep and shrinkage took place within the 373 day period.

Table 4.5 summarises the results from test series 1. It is seen from this table that the measured initial elastic shortening of the wallettes, $\varepsilon_{ni}$, was inconsistent with the applied force level for both ungrouted and grouted wallettes. This inconsistency was probably caused by the location of the measurement points on the base plates. It is likely that the base plates deformed during the stressing sequence, thus causing error to the measurements. This effect is not believed to have influenced measurements after application of axial load because the axial force in the wallettes remained nearly constant thereafter. Consequently, the creep coefficient and the specific creep given in Table 4.5 were based on the theoretical initial elastic shortening, $\varepsilon_{ni,t}$.

4.2.4 Series 2 experimental results

Fig. 4.7 shows the shortening strain (shrinkage + creep) curves for all wallettes with the two for the unstressed wallettes representing the pure shrinkage strain. Fig. 4.8 shows creep strain
TABLE 4.5—Series 1, summary of results

<table>
<thead>
<tr>
<th>Wall</th>
<th>Applied stress $f_{mi}$ (MPa)</th>
<th>Elastic shortening measured $e_{mi}$</th>
<th>Elastic shortening theoretical $e_{mi,t}$</th>
<th>Shortening under sustained load*</th>
<th>Creep Coeff. $C_C$</th>
<th>Specific creep $k_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-UG1</td>
<td>0.00</td>
<td>-</td>
<td>-</td>
<td>203</td>
<td>203</td>
<td>-</td>
</tr>
<tr>
<td>1-UG2</td>
<td>1.40</td>
<td>4</td>
<td>117</td>
<td>355</td>
<td>203</td>
<td>168</td>
</tr>
<tr>
<td>1-UG3</td>
<td>2.10</td>
<td>97</td>
<td>175</td>
<td>416</td>
<td>203</td>
<td>234</td>
</tr>
<tr>
<td>1-UG4</td>
<td>2.80</td>
<td>62</td>
<td>233</td>
<td>469</td>
<td>203</td>
<td>293</td>
</tr>
<tr>
<td>1-PG1</td>
<td>2.70</td>
<td>55</td>
<td>203</td>
<td>694</td>
<td>364</td>
<td>346</td>
</tr>
<tr>
<td>1-FG1</td>
<td>0.00</td>
<td>-</td>
<td>-</td>
<td>444</td>
<td>444</td>
<td>-</td>
</tr>
<tr>
<td>1-FG2</td>
<td>0.97</td>
<td>4</td>
<td>70</td>
<td>627</td>
<td>444</td>
<td>225</td>
</tr>
<tr>
<td>1-FG3</td>
<td>1.22</td>
<td>123</td>
<td>87</td>
<td>680</td>
<td>444</td>
<td>246</td>
</tr>
<tr>
<td>1-FG4</td>
<td>1.46</td>
<td>123</td>
<td>104</td>
<td>681</td>
<td>444</td>
<td>257</td>
</tr>
<tr>
<td>1-FG5</td>
<td>1.94</td>
<td>37</td>
<td>139</td>
<td>720</td>
<td>444</td>
<td>307</td>
</tr>
</tbody>
</table>

* maximum values, + based on $e_{mi,t} = f_{mi}/E_m$ where $E_m = 8000\mu$e

curves for all data sampled over 388 days. The creep results presented for grouted, partially grouted and un-grouted concrete masonry are not corrected for temperature and humidity variations throughout the sampling period.

Fig. 4.9 shows variation of the ambient air temperature and relative humidity measured continuously throughout the experiment (24 hour mean). It is noted that temperature and humidity measurements were taken every half hour throughout the test period and that the wallets were located in the Civil Test Hall basement. It is also noted that the ambient temperature at the beginning and end differed by about 2 degrees. Assuming a coefficient of thermal expansion for masonry of $10\mu$e/$^\circ$C a thermal expansion of $20\mu$e is calculated, suggesting that thermal expansion of the wallets did not affect the final strain readings significantly.

From Fig. 4.7 it appears that the shrinkage curves (unloaded wallets) started to level out after 150 days of measurements, and for the ungrouted wallets approximately levelled out. In a short time interval after day 220 all curves indicated expansion. This coincided with a rapid increase in relative humidity and temperature as shown in Fig. 4.9. As the relative humidity resumed normal levels of about 60%, the wallets resumed the shortening process. It is noted that the expansion of 2-FG5 (4.0 MPa) was significantly larger than that for the other fully grouted wallets. This abnormality was attributed to the splitting of 2-FG5 at day 220 which affected the subsequent measurements. The drop in strain of 2-FG5, just after day 220, of 160 microstrain should have been of the order of 80 microstrain, as was measured for the fully
grouted unstressed wallette 2-FG1. The true shortening and creep for 2-FG5 after day 220 should therefore be approximately 80 microstrain higher than shown in Figs. 4.7(a) and 4.8(a).

Considering Figs. 4.8(a) and (b), it is seen that the creep strain curves appear to have approximately levelled out by the end of the experiment. It is therefore assumed that most of the creep and shrinkage took place within the 388 day period.

From Series 2 test results, it would be reasonable to assume that the maximum shrinkage strains for un-grouted concrete masonry amount to approximately $450 \times 10^{-6}$ and for grouted concrete masonry amount to approximately $750 \times 10^{-6}$.

Table 4.6 summarises the results from test series 2. It is seen from this table that the measured initial elastic shortening of the wallettes, $e_{mi}$, was reasonably consistent with the applied force level. For this test series, there was good agreement between measured and estimated initial
Fig. 4.8—Series 2, creep strain

### TABLE 4.6—Series 2, summary of results

<table>
<thead>
<tr>
<th>Wall</th>
<th>Applied stress f_{mi}</th>
<th>Elastic shortening measured ( \varepsilon_{mi} )</th>
<th>Elastic shortening theoretical ( \varepsilon_{mi,t} )</th>
<th>Shortening under sustained load* ( \varepsilon_{sh} )</th>
<th>Total shortening ( \varepsilon_{sh} )</th>
<th>Creep ( \varepsilon_{cr} )</th>
<th>Creep Coeff. *( C_c )</th>
<th>Specific creep *( k_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-UG1</td>
<td>0.00</td>
<td>-</td>
<td>-</td>
<td>435</td>
<td>435</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2-UG2</td>
<td>1.14</td>
<td>126</td>
<td>95</td>
<td>741</td>
<td>435</td>
<td>321</td>
<td>3.39</td>
<td>282</td>
</tr>
<tr>
<td>2-UG3</td>
<td>2.14</td>
<td>208</td>
<td>178</td>
<td>852</td>
<td>435</td>
<td>430</td>
<td>2.43</td>
<td>202</td>
</tr>
<tr>
<td>2-UG4</td>
<td>2.99</td>
<td>261</td>
<td>249</td>
<td>1014</td>
<td>435</td>
<td>592</td>
<td>2.40</td>
<td>200</td>
</tr>
<tr>
<td>2-UG5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2-FG1</td>
<td>0.00</td>
<td>-</td>
<td>-</td>
<td>738</td>
<td>738</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2-FG2</td>
<td>1.00</td>
<td>91</td>
<td>69</td>
<td>950</td>
<td>738</td>
<td>220</td>
<td>3.19</td>
<td>219</td>
</tr>
<tr>
<td>2-FG3</td>
<td>1.82</td>
<td>118</td>
<td>125</td>
<td>1022</td>
<td>738</td>
<td>292</td>
<td>2.34</td>
<td>161</td>
</tr>
<tr>
<td>2-FG4</td>
<td>2.78</td>
<td>183</td>
<td>191</td>
<td>1091</td>
<td>738</td>
<td>361</td>
<td>1.89</td>
<td>130</td>
</tr>
<tr>
<td>2-FG5</td>
<td>4.00</td>
<td>283</td>
<td>274</td>
<td>1159</td>
<td>738</td>
<td>461</td>
<td>1.68</td>
<td>115</td>
</tr>
</tbody>
</table>

* maximum values, + based on \( \varepsilon_{mi,t} = f_{mi}/E_m \) where \( E_m = 800f'_{m} \)
Fig. 4.9—Series 2, ambient air temperature and humidity variations

elastic shortening values. To be consistent with Series 1, the creep coefficient and the specific creep given in Table 4.6 were based on the theoretical initial elastic shortening, $\varepsilon_{\text{mi},t}$.

As indicated in Table 4.4, 2-UG3, 2-UG4 and 2-FG5 split vertically down the middle after various durations under axial load. Only the splitting of 2-FG5 appears to have affected the creep results significantly, partly because of the late occurrence after about 230 days under load. The creep curves for 2-UG3 and 2-UG4 in Fig. 4.8(b) appear smooth and consistent.

4.2.5 Discussion of experiments

Shrinkage:

Comparing the measured shrinkage strain for the two test series, it is clear that the highest amount of shrinkage occurred in the 2nd test series. This can be explained by studying Figs. 4.6 and 4.9, showing the temperature and humidity values throughout the tests. It is clear that the
mean temperature was higher for the second test series which was stored in the Test Hall basement. The climate in the basement was clearly dryer than that of the Test Hall floor where testing series 1 was stored (humidity was not measured for series 1). Both higher temperature and lower humidity are expected to increase the drying shrinkage rate and also the long term shrinkage magnitude.

In fact, the environment in the Test Hall basement is considered to result in shrinkage strains exceeding those expected for walls in a realistic outdoor environment. This is because an outdoor environment (in New Zealand) is characterised by lower average temperature and higher relative humidity than those of the Civil Test Hall indoor environment. The maximum shrinkage strain measured in series 2 of $\varepsilon_{sh} = 450 \mu\varepsilon$ (microstrain) for ungrouted concrete masonry and $\varepsilon_{sh} = 750 \mu\varepsilon$ for grouted concrete masonry may therefore be considered as conservative.

When considering the shrinkage results it should be kept in mind that the measurements only include shrinkage recorded after application of prestress. Series 1 wallettes were grouted one day after construction, and the prestress was applied 11 days after construction. The series 2 wallettes were grouted 3 days after construction and stressed 10 days after construction. During this period the walls dried considerably. This is in particular the case for the grouted walls, which were heavily water saturated directly after the grouting process. In case of final application of prestress to a relatively wet wall a considerably higher amount of shrinkage strain should be expected.

The age of the masonry units at the time of wall construction also affects the anticipated shrinkage strain. Even though the masonry units have attained nearly all of the 28-day strength after perhaps 7 days, the drying shrinkage strain rate has not yet slowed down. In the extreme case it is possible that the masonry units are 7 days old at the time of construction and perhaps 14 days old at the time of final prestressing, hence in such situation a higher amount of shrinkage strain could be anticipated.

Creep:

Fig. 4.10 presents the experimentally derived creep coefficient $C_c$, calculated according to Eqn. 4.1 as a function of the axial force in the concrete masonry. The results are based on the modulus of elasticity defined by Eqn. 3.8 in Chapter 3. Note that this equation must be applied for estimation of $E_m$, when using the creep coefficients presented below. If the estimate of creep strain is based on specified strength, $E_m$ should be based on Eqn. 3.9. It is clear that compari-
son of experimental results and code-defined values may potentially be misleading because of significant differences in definition of $E_m$.

It is seen from Fig. 4.10 that there are significant differences in the results from the two test series. For ungrouted masonry (Fig. 4.10(a)) it is clear that smaller values were measured in series 1 and larger values measured in series 2. Also, the values of $C_c$ reduced with increase of $f_{mi}$, a maximum value of 3.4 being measured.

Regarding Fig. 4.10(b) for grouted concrete masonry it is seen that these test results showed a similar trend with reduction of $C_c$ for an increase of $f_{mi}$, but there was little scatter between the values from the two test series. The maximum value for $C_c$ for grouted concrete masonry recorded in these experiments was 3.2.

Fig. 4.11 shows the specific creep calculated with Eqn. 4.2 as a function of $f_{mi}$. These curves show similar trends to those of $C_c$ in Fig. 4.10. The maximum specific creep for ungrouted concrete masonry was $k_c = 282 \, \mu\varepsilon/\text{MPa}$ and for grouted concrete masonry $k_c = 231 \, \mu\varepsilon/\text{MPa}$.

Table 4.7 summarises the maximum values of the creep coefficient $C_{cc}$, specific creep $k_c$ and shrinkage strain $\varepsilon_{sh}$. These values should theoretically be amplified to account for the creep and shrinkage expected to occur between years 1 and 20. No magnification has been adopted herein because no research so far has indicated it's applicability to concrete masonry.
4.3 EXPERIMENTAL VS. CODE VALUES

4.3.1 Other experimental work

A compilation of recent experimental work on creep of concrete masonry was prepared by Hamilton and Badger [4-12]. The results were given in terms of $k_c$ and were specific to ungrouted concrete masonry. Values for light-weight and normal-weight units were presented and categorised with respect to the utilised mortar, M, S and N (refer to [4-4] for definition of mortar types), with typical mortar compressive strengths N: 5.2 MPa, S: 12.4 MPa and M: 17.2 MPa being adopted.

In New Zealand only one type of mortar is commonly used: Trade Mortar™. It’s a pre-mixed and pre-bagged product characterised by a specified strength of 12.5 MPa. The Australian designation for mortar of such properties is M3 or M4. In comparison with the US mortar designation, the New Zealand Trade Mortar™ can be classified as an S-mortar.

It is noted that the mortar type influences significantly the creep and shrinkage properties for ungrouted concrete masonry because much of the shortening takes place in the mortar joints.
The influence of mortar type on the creep and shrinkage of fully grouted concrete masonry is small because of the presence of the grout core that occupies much of the cross section.

Tables 4.8 and 4.9 show a compilation of research results, including the results found by experiment at the University of Auckland. Only results from experiments lasting more than 300 days are included. Results from the ‘Laursen’ experiment (University of Auckland) are associated with normal weight concrete masonry and S-mortar.

### 4.3.2 Code values for creep and shrinkage

Table 4.10 summarises values from creep and shrinkage provisions of BS 5628, AS 3700 and MSJC [4-2,4-3,4-4]. These provisions do not clearly distinguish between grouted and ungrouted masonry, so it is presumed that the values are primarily applicable to ungrouted concrete masonry. It should be noted that the calculation of creep strain due to $C_c$ is dependent on $E_m$, which is obtained in different ways by the individual codes.
4.3.3 Comparison

Creep:
It is seen from Table 4.8 that the range of $k_c$ values for ungrouted concrete masonry by Laursen, ranging from 105 $\mu\varepsilon$/MPa to 282 $\mu\varepsilon$/MPa, correspond reasonably well with those of Hamilton & Badger. The extreme value of 282 $\mu\varepsilon$/MPa was measured for 2-UG2 that was subjected to a relatively low axial load making the strain measurements less accurate. Furthermore, the Laursen values for grouted concrete masonry range somewhat higher than those measured for ungrouted masonry, from 115 $\mu\varepsilon$/MPa to 231 $\mu\varepsilon$/MPa. The relevance of the measurements by Laursen is thus confirmed. Table 4.9 shows that light-weight units generally result in less creep than normal weight units.

It is seen from Table 4.10 that the BS 5628 and AS 3700 values of $C_c$ are of same magnitude, but that the MSJC value is much lower. Comparison of the Laursen $C_c$ values to code values reveals that $C_c$ of the order of 3.0 is relevant. The MSJC value is unrealistic and is disregarded.

Shrinkage:
In terms of estimated shrinkage $\varepsilon_{sh}$, it appears that the BS 5628 and AS 3700 values of 500 $\mu\varepsilon$ and 700 $\mu\varepsilon$, respectively, are similar, but that the MSJC value is much lower. The values measured by Laursen for ungrouted concrete masonry of 203 $\mu\varepsilon$ to 435 $\mu\varepsilon$ are lower than the BS 5628 and AS 3700 values. For grouted concrete masonry higher shrinkage was measured, 444 $\mu\varepsilon$ for series 1 and 738 $\mu\varepsilon$ for series 2, values that fall near the BS 5628 and AS 3700 values.

4.4 RECOMMENDATIONS FOR CREEP AND SHRINKAGE

4.4.1 Creep
It is found that the creep coefficient $C_c$ based on experimental results for grouted concrete masonry is in agreement with the values stipulated by BS 5628 and AS 3700. For grouted concrete masonry it is acceptable to assume $C_c = 3.0$ for all prestress levels.

For ungrouted concrete masonry, it is clear that the values predicted by BS 5628 and AS 3700 are on the conservative side, especially for axial load levels higher than 2 MPa. The creep coefficient can safely be estimated at $C_c = 3.0$ for design purposes. The value of $C_c$ proposed by MSJC appears unrealistically low.
4.4.2 Shrinkage

Based on the experimental results and criteria provided in BS 5628 and AS 3700, it appears reasonable to use the following shrinkage strains for design under New Zealand conditions: For grouted walls $\varepsilon_{sh} = 700 \mu e$ and for un-grouted walls $\varepsilon_{sh} = 400 \mu e$. For ungrouted concrete masonry, the value is significantly lower than the value from AS 3700. For grouted concrete masonry the value is similar to the value from AS 3700. On the contrary, the value of $\varepsilon_{sh}$ proposed by MSJC appears unrealistically low.

4.4.3 Comments

It is noted that the results presented above were obtained under laboratory conditions. The masonry units were not sealed or surface treated, but all wallettes, including those un-stressed, were capped to simulate a typical construction scenario. Other factors such as storage conditions and age of the masonry units at the time of construction, were not taken into account. Prudence should be exercised when estimating creep and shrinkage losses for concrete masonry applied in extreme conditions.

The creep and shrinkage properties for partially grouted concrete masonry, which is becoming increasingly commonly used in New Zealand, should be based on interpolation of the results given in Table 4.11. Interpolation between the results for grouted and un-grouted concrete masonry may be done according to the ratio of grouted cells to the total number of cells.

The material parameter, $C_r$, should preferably be used for design, rather than $k_c$ which does not reflect actual material properties.

**TABLE 4.11—Recommended values of $C_r$, $k_c$, $\varepsilon_{sh}$.**

<table>
<thead>
<tr>
<th></th>
<th>$C_r$</th>
<th>$k_c$</th>
<th>$\varepsilon_{sh}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ungrouted</td>
<td>3.0</td>
<td>250</td>
<td>400</td>
</tr>
<tr>
<td>Grouted</td>
<td>3.0</td>
<td>250</td>
<td>700</td>
</tr>
</tbody>
</table>

4.5 TYPICAL PRESTRESS LOSS

The following section investigates typical trends using Eqn. 4.8, which is termed the ‘additive approach’ because of addition of shrinkage, creep and relaxation components without regard to interaction. Subsequently section 4.5.2 briefly explores an incremental approach for estimation of the total prestressing loss taking into account time variation and interaction between shrink-
age, creep and relaxation. That approach is termed ‘incremental approach’ and is compared with the additive approach.

4.5.1 Typical losses - additive approach

The magnitude of prestressing loss can be estimated using Eqn. 4.8 by considering typical material properties for concrete masonry and prestressing steel, and typical prestress and axial load levels. Table 4.11 gives the recommended values for calculation of prestress losses due to shortening of the concrete masonry. Long term relaxation after 50 years was taken as 2 times $k_r$ from Table 4.1.

Four cases are considered: (1) fully grouted wall prestressed with high strength prestressing strand, (2) fully grouted wall prestressed with high strength prestressing thread-bars, (3) ungrouted wall prestressed with high strength prestressing strand, and (4) ungrouted wall prestressed with high strength prestressing thread-bar. It is assumed that the crushing strength, $f_m^c$, for fully grouted masonry is 18 MPa and for ungrouted masonry is 15 MPa. Realistic axial load ratios, $\xi$, ranging from 0.05 to 0.30 are examined, with

$$\xi = \frac{P + N}{f_m^c l_w b_w} \quad (4.10)$$

Material properties for prestressing strand and thread-bar are taken from Table 3.6. It is furthermore acknowledged that the maximum achievable (permissible) prestress in the prestressing steel after lock-off is in the vicinity of 0.7$f_{pu}$ (NZS 3101 [4-20]).

The total prestress loss (long term loss) based on the above properties is represented in Figs. 4.12 and 4.13, defined as a percentage of the initial prestress for varying levels of axial load ratio. It is noted that the losses in terms of stress are independent of the actual prestressing force (kN) and that the axial force (N+P) can be constituted by any ratio of P vs. N. Prestressed retaining walls typically have low N, thus P amounts to nearly 100% of the axial load. Prestressed structural walls often are subjected to N larger than P, where N easily can amount to more than 50% of the axial load. It is also noted that if the axial force (N+P) has reduced significantly over time (P relatively large) an axial load based on the average of the initial and long term prestressing force may be used for estimation of the prestressing loss (iterative process).
It is seen from Fig. 4.12 that a high amount of loss can be expected for fully grouted concrete masonry. For the case of high strength strand with an initial stress of $0.7f_{pu}$ and an axial load ratio of 0.15, about 25% loss of prestress can be expected after all losses have occurred. For the case of a high strength bar solution with similar initial stress and axial load ratio a prestress loss of about 35% can be expected.
Fig. 4.13 shows that considerably less loss can be anticipated using ungrouted concrete masonry in comparison with fully grouted masonry. The main reason for the lower relative prestress loss for ungrouted masonry is that the anticipated shrinkage strain is lower than that of grouted masonry. For the case of high strength strand with an initial stress of $0.7f_{pu}$ and an axial load ratio of 0.15, about 20% loss of prestress can be expected after all losses have occurred. For a high strength bar solution, with similar initial stress and axial load ratio, a prestress loss of about 29% can be expected.

In both grouted and ungrouted concrete masonry, losses are considerable and must be taken into account in design. It is seen that lower initial stress levels result in higher relative losses. This is mainly the result of concrete masonry shrinkage strain becoming relatively more important at lower stress levels.

High prestress loss can be countered to some extent by restressing of the prestressing steel at some later time after the initial stressing. Figs. 4.4 and 4.7 suggest that approximately 50% of all creep and shrinkage takes place within 3 months. It is therefore in theory possible to reduce the prestress loss by about 50% if the restressing takes place about 3 months after the initial stressing.

Finally, if a prestressing steel stress (long term) of less than $0.7f_{pu}$ is required for ductility (drift capacity) considerations, it can in some cases be argued that application of an initial steel stress of $0.7f_{pu}$ is acceptable because creep, shrinkage and relaxation possibly will reduce the steel stress to the desired design level after, say, 3 months. Ductility requirements arise from seismic loading, which normally corresponds to the worst credible ground shaking with a return period of about 500 years. The risk of a seismic event during the period of ‘over-stress’ of three months (or even one year) is therefore negligible and likely to be acceptable.

### 4.5.2 Incremental approach

This section explores the total prestress loss when time effects have been taken into account. The term ‘incremental’ refers to evaluation of the total prestress loss by stepping through a finite number of time steps.

Initially, the time dependency of prestress losses due to shrinkage strain $\varepsilon_{sh}(t)$ and creep strain $\varepsilon_{cr}(t)$ are defined as [4-10] ($t$ in days):
\[ \Delta f_{cr}(t) = \varepsilon_{cr}(t)E_{ps} = F_{n,cr}(t) \frac{f_m(t)}{E_m} C_c E_{ps} = \frac{t^{0.6} f_m(t)}{10 + t^{0.6}} \frac{C_c E_{ps}}{E_m} \] (4.11)

\[ \Delta f_{sh}(t) = \varepsilon_{sh}(t)E_{ps} = F_{n,sh}(t) \frac{f_{sh}(t)}{E_{ps}} = \frac{t}{35 + t} \varepsilon_{sh} E_{ps} \] (4.12)

The prestress loss time dependency for high strength low relaxation strand is given by Eqn. 4.6 and repeated below with time dependency notation added (t in hours):

\[ \Delta f_{pr}(t) = f_{ps}(t) \left[ \frac{Log_{10}(t)}{45} \left( \frac{f_{ps}(t)}{f_{py}} - 0.55 \right) \right] \text{ for } \frac{f_{ps}(t)}{f_{py}} \geq 0.55 \] (4.13)

It is noted from Eqns. 4.11 and 4.13 that prestress loss due to creep and relaxation not only are functions of direct time variation \( F_n(t) \) but also function of the instantaneous masonry stress \( f_m(t) \) and prestress \( f_{ps}(t) \). Inherent in Eqns. 4.11 and 4.13 is the assumption that any increment in prestress loss due to creep and relaxation is a direct function of the instantaneous masonry stress \( f_m(t) \) and instantaneous prestress \( f_{ps}(t) \) and not the initial masonry stress \( f_{mi} \) and initial prestress \( f_{psi} \). The shrinkage rate is per definition independent of the masonry stress.

Fig. 4.14 shows \( F_n(t) \) for shrinkage, creep and relaxation for time between zero and 1000 days (approximately 3 years). It is seen that the shrinkage rate is higher than the creep rate and that relaxation initially occurs at a higher rate than shrinkage and creep.
The total prestress loss at any instant can be written as:

\[
\Delta f_{pl}(t) = F_n\{ \Delta f_{cr}(t), \Delta f_{pl}(t), \Delta f_{sh}(t), \Delta f_{pr}(t, \Delta f_{pl}(t)) \}
\]  

(4.14)

which indicates a complex relationship with \( \Delta f_{pl}(t) \) occurring on both sides of the equation. This equation can be approximately solved numerically, e.g. assuming that the masonry stress \( f_m(t) \) and the prestress \( f_{ps}(t) \) remain constant in small time intervals: \( t_n \) to \( t_{n+1} \). Within each time interval (step) \( \Delta t_n = t_{n+1} - t_n \), the creep, shrinkage and prestress relaxation increments are defined as follows:

\[
\Delta f_{cr}(\Delta t_{n+1}) = [F_n^{cr}(t_{n+1}) - F_n^{cr}(t_n)] \frac{f_m(t_n)}{E_m} C P E_{ps}
\]

(4.15)

\[
\Delta f_{sh}(\Delta t_{n+1}) = [F_n^{sh}(t_{n+1}) - F_n^{sh}(t_n)] \varepsilon_{sh} E_{ps}
\]

(4.16)

\[
\Delta f_{pr}(\Delta t_{n+1}) = \frac{f_{ps}(t_n)}{45 \log_{10} \left( \frac{t_{n+1}}{t_n} \right) \left( \frac{f_{ps}(t_n)}{f_{py}} - 0.55 \right)}
\]

(4.17)

where \( \frac{f_{ps}(t_n)}{f_{py}} \geq 0.55 \)

From Eqns. 4.15, 4.16 and 4.17 it is found that:

\[
P(t_{n+1}) = P(t_n) - [\Delta f_{cr}(\Delta t_{n+1}) + \Delta f_{sh}(\Delta t_{n+1}) + \Delta f_{pr}(\Delta t_{n+1})] A_{ps}
\]

(4.18)

\[
f_{ps}(t_{n+1}) = \frac{P(t_{n+1})}{A_{ps}}
\]

(4.19)

\[
f_m(t_{n+1}) = \frac{N + P(t_{n+1})}{A_g} \text{ where } A_g = l_w b_w
\]

(4.20)

The solution to time step \( n+1 \) is thus solely based on the previous time step \( n \) and the given functions of \( F_n(t) \). The total prestress loss is evaluated, using the initial conditions: \( f_m(0) = f_{mi} \) and \( f_{ps}(0) = f_{psi} \), by stepping through the Eqns. 4.15 to 4.20 a total of \( M \) times. As the time step length diminishes (number of increments \( M \) increases), the solution converges to the exact solution.

Comparison between the incremental and additive approaches may be found in Fig. 4.15. Prestress loss after 50 years was calculated according to Eqn. 4.8 (additive approach) and Eqn. 4.18 (increments approach) using the following input: \( f_m = 18 \) MPa, \( E_m = 14.4 \) GPa, \( \varepsilon_{sh} = \ldots \)
Fig. 4.15—Comparison of prestress loss, incremental vs. additive approaches

0.000700, $C_c = 3.0$, $f_{psi} = 0.7f_{pu}$, $f_{pu} = 1750$ MPa, $f_{py} = 1488$ MPa, $E_{ps} = 190$ GPa and $k_r(50$ years) = 0.034. The axial load ratio $\xi$ was varied in the interval 0.05 to 0.25 and the ratio of initial prestressing force to total initial axial force $\Lambda = P_i/(P_i+N)$ was varied from 0.1 (very light prestressing) to 1.0 (axial force purely due to prestressing). A constant time step of 1 day was used for the calculation.

Fig. 4.15 displays a series of plots of the ratio between total prestress loss calculated with the additive and incremental approaches. This ratio is termed ‘Loss ratio’. For a given axial load ratio, the loss ratio was calculated for a series of prestress force to total force ratios $\Lambda$. In essence, the figure can be interpreted as the relative benefit of conducting prestress loss assessment using the more cumbersome and presumably more accurate incremental method.

The following observation are made from Fig. 4.15: (1) the loss ratio reduces with increasing $\Lambda$, (2) the loss ratio reduces with increasing $\xi$ for $\Lambda$ greater than about 0.3, (3) the prestress loss calculated using the incremental method is at least 10% less than that calculated with the additive method and (4) at the best predicts about 15% less prestress loss, and (5) a loss ratio of 0.88 is found for typical PCM wall values $\Lambda = 0.5$ and $\xi = 0.15$.

In conclusion, the incremental method gives favourable results when compared with the additive method in terms of structural efficiency and economy. A typical reduction in predicted pre-
stress loss of 12% can be expected using the incremental method. Given the small magnitude of achieved improvement using the incremental method in comparison with the significant uncertainty of the shrinkage and creep properties for concrete masonry, it is questionable whether use of the incremental method can be justified. It is therefore recommended to use the additive method for prediction of prestress losses in PCM walls as defined by Eqn. 4.8. This section confirms that typical prestress losses as calculated and discussed in section 4.5.1 are relevant to typical PCM wall design.
4.6 REFERENCES


[4-10] ACI Special Publication - 76, Designing for Creep & Shrinkage in Concrete Structures, Published by American Concrete Institute, 1982, 484p.


Chapter 5

STRUCTURAL TESTING - SERIES 1

5. SINGLE-STOREY PCM WALLS

In-plane response of post-tensioned concrete masonry walls with unbonded tendons is examined by means of structural testing. A description of the Series 1 wall testing programme is followed by a presentation of the results from structural testing of six fully grouted walls, one partially grouted wall and one ungrouted wall. Discussion of the results is concerned with wall structural response in terms of flexural strength, shear strength, displacement capacity, tendon stress and masonry vertical strain.

5.1 INTRODUCTION

This chapter describes the results from structural testing of eight unbonded PCM cantilever walls. The walls were tested to explore the behaviour of single-storey rectangular post-tensioned walls of various aspect ratio, thickness and tendon layout. The objective of this study was to obtain data for use in design; however, design examples are not considered herein.

Using unbonded post-tensioning, walls are vertically prestressed by means of strands or bars which are passed through vertical ducts inside the walls. As the walls are subjected to lateral in-plane displacements, gaps form at the horizontal joints reducing the system stiffness. As long as the prestressing strands retain a significant force they will return the walls to their initial position. Thus, the lateral force-displacement response may be described by a nearly nonlinear elastic relationship. The integrity of the walls is maintained as no plastic hinges form and there are no residual post-earthquake displacements. Consequently minimal structural and non-structural wall damage can be expected.
5.2 CONSTRUCTION DETAILS

5.2.1 Wall specifications

Wall dimensions are specified in Table 5.1. All walls were 2.6 m high. The wall designations may be deciphered in the following way: FG indicates fully grouted, PG indicates partially grouted, UG indicates ungrouted, L3.0 indicates that the wall was 3.0 m long, W15 indicates a nominal wall thickness of 150 mm and P3 means that three prestressing bars were embedded in the wall.

FG:L3.0-W20-P3 was the only wall constructed of 20 series masonry units and was the only wall that had horizontal shear reinforcement embedded. All other walls were constructed of 15 series or 10 series concrete masonry units, without shear reinforcement. The actual thickness of the walls was approximately 10 mm less than the nominal masonry unit thickness. The partially grouted wall had the end flues and two intermediate flues grouted; all other flues remained unfilled.

All walls were prestressed with two or three 23 mm VSL thread bars. For the long walls (L3.0) with three bars, two were placed 300 mm from the wall ends and the other was placed on the wall centreline. FG:L1.8-W15-P3 had two bars placed concentrically at ±400 mm from the wall centreline and the last bar placed on the wall centreline. For the case of walls with two bars, FG:L3.0-W15-P2C, FG:L1.8-W15-P2 and UG:L1.8-W10-P2 had bars placed concentrically at ±400 mm from the wall centreline, while FG:L3.0-W15-P2E and PG:L3.0-W15-P2 had bars placed eccentrically at 300 mm from the wall ends. The prestressing tendons remained unbonded over the entire wall height and had a typical unbonded length of 3.5 m.

<table>
<thead>
<tr>
<th>Wall</th>
<th>length $l_w$ (m)</th>
<th>thickness $b_w$ (mm)</th>
<th>self weight $N$ (KN)</th>
<th>masonry strength $f'_{m}$ (MPa)</th>
<th>tendons</th>
<th>initial steel stress $f_{ps}$ (MPa)</th>
<th>initial prestress force $P$ (KN)</th>
<th>axial force ratio $f_m/f'_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. FG:L3.0-W20-P3</td>
<td>3.0</td>
<td>190</td>
<td>50</td>
<td>13.3</td>
<td>3</td>
<td>468</td>
<td>580</td>
<td>0.083</td>
</tr>
<tr>
<td>2. FG:L3.0-W15-P3</td>
<td>3.0</td>
<td>140</td>
<td>41</td>
<td>15.1</td>
<td>3</td>
<td>555</td>
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<tr>
<td>3. FG:L3.0-W15-P2C</td>
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<tr>
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<td>17.8</td>
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<td>628</td>
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<td>5. FG:L1.8-W15-P2</td>
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<td>445</td>
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<tr>
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<td>18.4</td>
<td>3</td>
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<td>760</td>
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</tr>
<tr>
<td>7. FG:L3.0-W15-P2</td>
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<td>15.5</td>
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<td>146</td>
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<tr>
<td>8. UG:L1.8-W10-P2</td>
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<td>308</td>
<td>256</td>
<td>0.162</td>
</tr>
</tbody>
</table>

TABLE 5.1—Wall specifications
including unbonded lengths in the foundation (300 mm), loading beam (400 mm) and load cells on top of the loading beam (200 mm). The wall prestressing force and axial force ratios (due to prestressing and self-weight) are given in Table 5.1.

5.2.2 Wall construction

Prior to wall construction, the prestressing bars were inserted in the foundation and encased by ducting to ensure proper isolation from the grout. The walls were constructed in running bond by experienced blocklayers under supervision, using standard grey precast concrete masonry units and DRICON™ trade mortar. Open-ended concrete masonry units were used for most of the walls in order to avoid having to thread the masonry units onto the 4 m long prestressing bars. Grouting with 17.5 MPa ready-mixed regular block fill, containing SIKA CAVEX™ expansion agent, was performed the subsequent day.

5.2.3 Material properties

Masonry crushing strength was determined by material testing of masonry prisms (height by length of 600 mm by 400 mm) for walls FG:L3.0-W15-P2C, FG:L1.8-W15-P2, FG:L1.8-W15-P3 and UG:L1.8-W10-P2. Strengths of walls FG:L3.0-W20-P3, FG:L3.0-W15-P3, FG:L3.0-W15-P2E and PG:L3.0-W15-P2 were estimated on a statistical basis from knowledge of masonry block, mortar and grout strengths.

VSL 23 mm diameter high strength threaded bars were used in all wall tests and had the typical properties: yield strength of 970 MPa, ultimate strength of 1160 MPa, ultimate strain of 8% and modulus of elasticity of 190 GPa.

5.3 TESTING DETAILS

5.3.1 Test setup

Wall testing reported herein was conducted using the setup shown in Fig 5.1(a), consisting of a reusable foundation and loading beam. Notably, no effort was made to replicate gravity and live loads from suspended floors, with the assumption that these loads were directly analogous to a component of the applied post-tensioning force. Fig. 5.1(b) schematically shows typical wall instrumentation. Lateral force was measured by two load cells positioned in series with the hydraulic actuators, denoted as instruments 1 and 2 in Fig. 5.1(b). Wall flexural deformation and vertical strain were measured by instruments 5-18 (portal type LVDTs) and panel
deformation was measured by instruments 19-38. Relative sliding displacements between loading beam/wall, wall/foundation and foundation/strong floor were measured by instruments 39, 40 and 41, respectively.

5.3.2 Testing procedure

Cyclic structural testing was conducted according to the procedure outlined by Park [5-1], with the ductility one displacement here termed 'displacement level one' or D1. After definition of D1, the walls were cycled to D2, D4, D6, etc. until failure, with Dn being defined as n times D1. Prior to executing this loading procedure, serviceability limit state level force cycles were applied to some walls in order to investigate the uncracked stiffness, the lateral displacement profile and the initial crack formation.

5.3.3 Predicted nominal flexural strength

For a symmetric distribution of post-tensioning and gravity actions, the nominal flexural strength, $M_n$, is defined by the New Zealand Masonry code NZS 4230:1990 [5-2] as given in Eqns. 8.17, 8.18 and 8.29. Details of these equations can be found in Chapter 8. The base shear, $V_r$, corresponding to $M_n$ may be calculated using Eqn. 8.17, where $P$ represents the initial prestress force (no lateral load applied to wall), $\Delta P$ is the total tendon force increase due to wall deformation at $V_r$, $l_w$ is the wall length, $h_c$ is the height of the wall and $N$ is the axial load at the base of the wall due to live and dead load. The equivalent rectangular compression zone length, $a$, is defined by Eqn. 8.18 as a function of the axial forces on the section, the masonry compressive strength, $f_{m}'$, the wall width $b_w$ and $\alpha = 0.85$ (unconfined grouted concrete masonry). Eqn. 8.29 defines the effective tendon force eccentricity, $e$, with respect to the initial
prestressing force and the tendon force increase based on the assumption of uniformly distributed prestressing steel. In the absence of suitable procedures for estimating \( \Delta P \) at the time of testing, the tendon force increase at nominal strength was estimated to be 50 kN for the fully grouted walls and zero for the partially and ungrouted walls. The latter assumption of zero force increase was based on expectation of minor flexural deformation for these wall types.

### 5.3.4 Predicted masonry shear strength

The predicted masonry shear strength (stress) was calculated according to the following formulae: Eqn. 5.1 - NEHRP [5-3], Eqn. 5.2 - Paulay and Priestley [5-4] and Eqn. 5.3 - NZS 4230:1990, using the notation adopted herein.

\[
v_m = 0.083 \left( 4 - 1.75 \frac{h_u}{t_w} \right) \sqrt{j_m'} + 0.25 \frac{N + P}{A_g} \leq 0.33 \sqrt{j_m'} \tag{5.1}
\]

\[
v_m = 0.136 \sqrt{j_m'} + 0.24 \frac{N + P}{A_g} \leq 1.04 \text{ MPa} \tag{5.2}
\]

\[
v_m = 0.024 \sqrt{j_m'} + 0.24 \frac{N + P}{A_g} \leq 0.576 \text{ MPa} \tag{5.3}
\]

The masonry shear strength in terms of force, \( V_s \), is calculated by multiplication of \( v_m \) by the gross cross-sectional area, \( A_g \), for fully grouted walls. For the partially and ungrouted walls, due consideration was made for ungrouted cavities (refer to section 5.2). The predicted masonry shear strengths, \( V_s \), listed in Table 5.2 were based on Eqn. 5.1, this equation being most recently developed. The initial prestress force was used in this calculation.

### TABLE 5.2—Predictions and results

<table>
<thead>
<tr>
<th>Wall</th>
<th>Predictions</th>
<th>Test results</th>
<th>behaviour</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>( V_f )</td>
<td>( V_s )</td>
<td>( V_{\text{max}} )</td>
</tr>
<tr>
<td>1. FG:L3.0-W20-P3</td>
<td>336</td>
<td>567</td>
<td>561</td>
</tr>
<tr>
<td>2. FG:L3.0-W15-P3</td>
<td>367</td>
<td>504</td>
<td>465</td>
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<tr>
<td>3. FG:L3.0-W15-P2C</td>
<td>356</td>
<td>542</td>
<td>373</td>
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<td>4. FG:L3.0-W15-P2E</td>
<td>356</td>
<td>515</td>
<td>373</td>
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<td>5. FG:L1.8-W15-P2</td>
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<td>332</td>
<td>178</td>
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<td>6. FG:L1.8-W15-P3</td>
<td>218</td>
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<td>266</td>
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<td>7. FG:L3.0-W10-P2</td>
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<td>180</td>
<td>120</td>
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<td>8. UG:L1.8-W10-P2</td>
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<td>102</td>
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<td>kN</td>
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<td>mm</td>
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</table>
5.4 TEST RESULTS

This section describes the behaviour of the eight walls and presents the measured masonry compression toe strains at various wall states. General wall behaviour is summarised in Table 5.2, where \( V_{\text{max}} \) is the maximum lateral force recorded and \( d_{\text{vmax}} \) is the corresponding displacement. The ultimate displacement capacity, \( d_u \), is defined as the point at which the lateral wall strength had degraded below 80\% of \( V_{\text{max}} \). The ultimate drift capacity is defined as \( \gamma_u = d_u/h_c \).

5.4.1 Fully grouted walls

Rocking response was recorded for all fully grouted wall tests, characterised by opening of a single large crack along the wall-foundation construction joint. No distributed flexural cracks were observed and distinct diagonal compression strut cracking was only observed for FG:L1.8-W15-P3. High initial stiffness was recorded. At nominal flexural strength, \( M_n \), wall softening occurred due to the initiation of rocking. Tendon yielding was experienced for all walls that failed in flexure.

The failure mode was characterised by localised masonry crushing in the compression toe regions, resulting in gradual strength degradation (with the exception of wall FG:L1.8-W15-P3 that failed in shear).

FG:L1.8-W15-P3 exhibited rocking response for displacements up to approximately D1 (4.3 mm). Displacing the wall towards D2 (8.6 mm) initiated diagonal cracking, eventually leading to failure in both directions of loading.

5.4.1.1 Force-displacement response

Fig. 5.2, depicting the force-displacement response for all fully grouted walls, demonstrates the nearly non-linear elastic behaviour exhibited by unbonded post-tensioned walls prior to toe crushing. It is seen that the walls returned to their original alignment, even after large displacement excursions. Therefore the response may be categorised as “origin oriented”. The individual curves for each excursion appear pinched, implying little hysteretic energy dissipation. It is noted that the term ‘elastic’ in a rigorous sense indicates reversibility and no accumulation of damage. In reality the walls did accumulate damage from the onset of tendon yielding and masonry crushing, causing a reduction in wall stiffness. Thus the behaviour was termed ‘nearly’ non-linear elastic.
All walls, except for FG:L3.0-W15-P3 and FG:L3.0-W15-P2E, exhibited nearly symmetrical response in the push and pull directions. FG:L3.0-W15-P3 was accidentally displaced -34 mm, as shown in Fig. 5.2(b), causing serious damage to the wall and preventing effective further testing in the pull direction. The asymmetric behaviour of FG:L3.0-W15-P2E, seen in Fig. 5.2(d), was caused by extensive sliding. Note that the data depicted in Fig. 5.2(d) was corrected for sliding displacement, thus representing only the wall flexural and shear deformation behaviour.

Fig. 5.2—Fully grouted walls; Force-displacement histories
Gradual strength degradation was observed for all FG walls (except for FG:L1.8-W15-P3), attributed to spalling of face shells and crushing of the grout core. Large displacement capacity was observed with reliable wall resistance for maximum drifts ranging from 0.73% to more than 1.4%, refer to Table 5.2. For all FG walls except FG:L1.8-W15P3, the crack patterns after failure, depicted schematically in Fig. 5.3, indicated that the extent of damage was confined to
the lowest two masonry courses in the toe regions. The total lack of shear cracking for FG:L3.0-W20-P3 indicated that the shear reinforcement was not engaged at all, explaining the omission of this reinforcement from all subsequent tests in this series.

Fig. 5.3(f) shows the final crack pattern for the only FG wall that failed in compression strut splitting, FG:L1.8-W15-P3. The failure mode was characterised by diagonal cracking due to tensile splitting of the masonry compression struts forming between the prestressing anchorage locations in the loading beam and the wall flexural compression zone. It is seen in Fig. 5.2(f) that the wall strength exceeded $V_f$ before failure, thus indicating a brittle failure mode influenced by flexure. This type of failure was unexpected as the predicted masonry shear strength was higher than the predicted flexural strength, as indicated in Table 5.2.

Fig. 5.3(b) reveals an inclined crack that appeared while pulling FG:L3.0-W15-P3 to -D4 (-12 mm). This crack is believed to have been caused by large localised splitting forces associated with the prestress anchorage at the centre of the loading beam. The crack did not develop further during testing, possibly due to transverse restraint provided by the loading beam, and did not appear to affect the remainder of the test.

5.4.1.3 Sliding

There was no indication of wall sliding relative to the foundation for FG:L3.0-W20-P3, FG:L3.0-W15-P3 and FG:L1.8-W15-P3. Walls FG:L3.0-W15-P2C and FG:L1.8-W15-P2 showed sliding displacements of less that 5% of the total displacement, while sliding of more than 17 mm was measured in the pull direction for FG:L3.0-W15-P2E. Sliding became more pronounced for the last walls tested. This was because of gradual deterioration of the foundation beam roughness as the test series progressed, thus reducing the shear friction along the wall/base construction joint.

5.4.1.4 Prestressing force

The prestressing tendon force histories (sum of force in three tendons) for all FG walls are plotted against lateral displacement in Fig. 5.4. It is seen that the first significant total tendon force loss (more than 5% of the initial force) for the walls that failed in flexure occurred after lateral wall displacements ranging from $\pm 8$ mm to $\pm 12$ mm, in all cases roughly corresponding to first yield of the extreme tendons when considering the wall as a rigid body rocking about the wall corners. At the conclusion of testing, between 25% and 75% of the initial total pre-
stressing force remained. Fig. 5.4(f) shows the prestressing tendon force history for FG:L1.8-W15-P3 and reveals no prestressing force loss before failure.

5.4.2 Partially and ungrouted walls

These walls clearly exhibited shear dominated response, signified by development of shear cracks spanning diagonally across the wall face. Flexural action was observed at force levels
Fig. 5.5—Partially and ungrouted walls; Force-displacement histories

up to approximately nominal flexural strength, $V_f$, as indicated in Fig. 5.5. Distress of the flexural compression zone was not observed and tendon yielding never occurred.

5.4.2.1 Force-displacement response

Fig. 5.5(a) shows the force-displacement history for PG:L3.0-W15-P2. At displacements of approximately $+3.9/-2.3$ mm, diagonal shear cracking initiated with significant strength degradation in both directions of loading, however PG:L3.0-W15-P2 did not exhibit sudden failure at this point and retained more than 60% of its maximum strength up to displacements of approximately $\pm 10$ mm.

Wall UG:L1.8-W10-P2 initially exhibited flexural and rocking response, signified by opening of one single crack at the base of the wall for load levels up to $V_f$. Inclined shear cracking then initiated, resulting in wall failure. Fig. 5.5(b) shows the non-linear elastic wall response of UG:L1.8-W10-P2 with nearly symmetrical response in the push and pull directions for applied lateral forces up to $V_f$.

5.4.2.2 Damage pattern and failure mode

The failure modes for both the PG and UG walls were characterised by diagonal shear cracking initiated by tension splitting along the axis of the diagonal strut. For the PG wall, the cracks followed the vertical and horizontal mortar joints as shown in Fig. 5.6(a). For the UG wall the cracks took a more direct path through the concrete masonry units, as shown in Fig. 5.6(b). This failure mode was anticipated for both walls, despite prediction of nominal shear strength higher than nominal flexural strength (Table 5.2), because the relatively high steel content was
Section of partially and ungrouted walls; Extent of damage at failure
predicted to develop significant flexural overstrength, thus exceeding the nominal shear strength.

5.4.2.3 Sliding

Sliding was recorded in both wall tests, although the sliding displacement at any given stage amounted to less than 5% of the total displacement for the PG wall and to less than 7% of the total displacement for the UG wall.

5.4.2.4 Prestressing force

Significant tendon force increase due to flexural action was measured during testing of PG:L3.0-W15-P2, prior to the onset of strength degradation, but tendon force eccentricity was determined to be insignificant. Testing of UG:L1.8-W10-P2 showed little prestressing force variation and the associated tendon force eccentricity was insignificant compared with the wall length.

5.4.3 Vertical masonry strain

Vertical strains in the flexural compression zones (the toe) of the walls were evaluated from data acquired during testing for the following wall states: (1) at nominal flexural strength, $V_f$, (2) at the peak of the excursion causing the first signs of wall toe distress, (3) at maximum strength, $V_{max}$, and (4) at ultimate displacement, $d_u$. It is noted that (1) was based on Eqns. 8.17, 8.18 and 8.29 and (2) was based purely on visual inspection. Refer to Table 5.2 for $V_f$, $V_{max}$ and $d_u$. The measured strains in the extreme compression fibre at the wall states described above are listed in Table 5.3 as the average for the push and pull directions, as available. The strains were measured in the lowest 200 mm of the walls.
5.5 DISCUSSION

This section discusses how design parameters, such as height, length, prestressing and masonry properties affect the wall behaviour in terms of flexural strength, displacement capacity, tendon stress, masonry vertical strain, shear strength and sliding propensity. Selection of appropriate axial load ratio is also discussed.

5.5.1 Fully grouted walls

5.5.1.1 Flexural strength

The wall flexural strength (maximum moment) is dependent on the wall dimensions, in particular wall length $l_w$. This is illustrated in Fig. 5.7, which shows the force-displacement envelopes (FDE) for all walls. Walls FG:L1.8-W15-P2 and FG:L3.0-W15-P2C, which had similar parameters except for wall length, clearly showed that wall strength increased from about 185

<table>
<thead>
<tr>
<th>Wall</th>
<th>$V_f$</th>
<th>$V_{max}$</th>
<th>$d_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1: FG:L3.0-W20-P3</td>
<td>0.0025</td>
<td>0.0137</td>
<td>0.0253</td>
</tr>
<tr>
<td>2: FG:L3.0-W15-P3</td>
<td>0.0027</td>
<td>0.0104</td>
<td>0.0228</td>
</tr>
<tr>
<td>3: FG:L3.0-W15-P2C</td>
<td>0.0039</td>
<td>0.0104</td>
<td>0.0168</td>
</tr>
<tr>
<td>4: FG:L3.0-W15-P2E</td>
<td>0.0027</td>
<td>0.0104</td>
<td>0.0112</td>
</tr>
<tr>
<td>5: FG:L1.8-W15-P2</td>
<td>0.0033</td>
<td>0.0224</td>
<td>0.0192</td>
</tr>
<tr>
<td>6: FG:L1.8-W15-P3</td>
<td>0.0031</td>
<td>0.0104</td>
<td></td>
</tr>
<tr>
<td>7: FG:L3.0-W15-P2</td>
<td>0.0016</td>
<td>0.0104</td>
<td></td>
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<tr>
<td>8: FG:L1.8-W10-P2</td>
<td>0.0018</td>
<td>0.0104</td>
<td></td>
</tr>
</tbody>
</table>

TABLE 5.3—Measured masonry vertical strain
kN to approximately 367 kN. The effect of wall thickness on wall strength is illustrated by comparison of the FDEs for FG:L3.0-W20-P3 and FG:L3.0-W15-P3. It is seen that for these walls, having the same quantity of prestressing steel, the thicker wall (W20) ultimately developed higher strength than the thin wall, ca. 549 kN vs. 472 kN.

The effect of the reinforcing ratio (area of prestressing steel, $A_{ps}$, relative to the gross sectional area, $A_g$: $\rho_{ps} = A_{ps}/A_g$) on wall strength can be illustrated by comparison of walls FG:L3.0-W15-P3 and FG:L3.0-W15-P2C, which share the same length and thickness. Similarly walls FG:L1.8-W15-P2 and FG:L1.8-W15-P3 may be compared. From Fig. 5.7 and Table 5.2 it is clear that the walls with the highest reinforcing ratio (3 prestressing tendons) developed the highest strength.

Fig. 5.8 shows the wall FDEs normalised with predicted nominal flexural strength in terms of base shear, $V_f$. It is noted that the actual wall strengths for all walls exceeded the nominal strength with a margin of 5% to 66%. Walls with low initial prestressing tendon stress, e.g. FG:L3.0-W20-P3, developed the largest overstrength, emphasising the significance of tendon reserve elastic capacity.

5.5.1.2 Ultimate drift capacity

The wall ultimate drift capacity, $\gamma_\omega$, is dependent on wall dimensions. An increase of wall length is likely to reduce the displacement capacity because a longer wall tends to develop significant tendon force increase at a smaller displacement. This tendon force increase is then likely to promote distress of the masonry compression zone at a lower displacement. Comparison of wall tests FG:L3.0-W15-P2C and FG:L1.8-W15-P2 in Fig. 5.8 suggests that for walls...
with identical $\rho_{ps}$ and location of prestressing steel, the shorter wall had considerably larger ultimate displacement capacity. Increase of wall height is expected to increase the wall ultimate displacement capacity, partially due to the longer unbonded length of the prestressing tendons. The ultimate drift capacity thus increases with an increase in wall aspect ratio, $A_r = h/w$.

An increase of reinforcing ratio, $\rho_{ps}$, is expected to reduce the wall ultimate drift capacity as a consequence of an increase of the flexural compression zone length, which imposes higher strain on the extreme masonry fibre at any given wall displacement. This behaviour is illustrated by comparison of walls FG:L3.0-W15-P3 and FG:L3.0-W15-P2C, which share the same masonry dimensions. The initial prestress in the tendons is not expected to influence the wall ultimate displacement capacity greatly because $\gamma_u$ is rather controlled by the magnitude of the tendon yield force.

### 5.5.1.3 Tendon yielding

Tendon yielding was generally expected for all walls because of the relatively short unbonded tendon lengths, and was recorded in all tests except FG:L1.8-W15-P3. The wall drift corresponding to tendon yield is largely affected by three parameters, wall aspect ratio, $A_r$, tendon initial stress and tendon location.

A longer wall typically results in a lower tendon yield displacement because of comparatively larger tendon strains at a given lateral displacement. An increase in wall height results in longer unbonded tendon length and therefore in higher tendon elongation and base rotation before yield. Thus an increase of the wall aspect ratio, $A_r$, results in higher wall drift before tendon yield, assuming that the tendons remain unbonded over the entire wall height.

In order to effectively avoid yielding of the prestressing tendons, it is typically necessary to keep the tendons unbonded over several storeys and to keep the initial tendon stress relatively low, say in the range of 40%-60% of the tendon yield stress. In typical masonry structures, it may not be feasible to require elastic tendon response at high displacements. Still, adequate wall performance can be achieved despite yielding of tendons, as has been demonstrated by the research described herein. It is noted that perhaps 50% the restoring moment for typical structural walls originates from the selfweight of the wall and supported floors. A parallel can be drawn to research by Kurama et al. [5-5], who investigated unbonded post-tensioned concrete
walls and concluded that adequate seismic performance could be achieved despite yielding of tendons.

Also, the tendon distribution along the length of the wall significantly affects the tendon yield displacement. Considering the two extreme tendon locations, at the wall ends and at the wall centreline, it becomes evident that the strain in the tendon on the wall centreline is approximately half of the strain in the tendon at the wall end for a given lateral displacement, assuming wall rocking about the corners. It may not be feasible for masonry walls to have all tendons placed near the wall centreline. Use of distributed tendons may be necessary due to consideration of e.g. wall penetrations and out-of-plane forces.

5.5.1.4 Vertical masonry strain

Table 5.3 shows the vertical strain recorded in the walls at different stages of the tests. It is seen that the calculated masonry strains corresponding to the nominal flexural condition ranged between 0.0025 and 0.0039. These results suggest that the above equations capture the code-defined nominal flexural strength condition in a realistic and reasonably consistent fashion. An average strain of 0.0030 was found when discarding the two extreme values. This result can be compared with the strain limit at nominal flexural strength (unconfined masonry) given by the New Zealand masonry code, NZS 4230:1990, of 0.0025, refer to Chapter 3.

Masonry strains associated with the first sign of toe region distress may be regarded as an alternative definition of the ultimate flexural condition. It is seen from Table 5.3 that these strain values vary between 0.0067 and 0.0104, about 2 to 3 times higher than those associated with nominal flexural strength.

Masonry strains associated with the maximum wall strength were for walls FG:L3.0-W15-P2C and FG:L3.0-W15-P2E similar to those associated with the first visual sign of toe distress. Walls FG:L3.0-W15-P3 and FG:L1.8-W15-P2 showed that the masonry strains at maximum strength were three to four times as high as those associated with the first visual sign of toe distress.

Table 5.3 indicates that masonry extreme fibre strains ranged between 1.1% and 2.5% at the ultimate displacement. These values are of course of theoretical character as the toe regions were damaged at this stage, so that those strains should not be regarded as the reliable ultimate strain capacity of concrete masonry. The results do nevertheless suggest that masonry strains
far beyond those related to nominal flexural strength can be expected for rocking wall systems, while still providing significant axial strength.

5.5.1.5 Masonry shear strength

As suggested by the test results and Eqns. 5.1 to 5.3, the masonry shear strength is greatly enhanced by the additional axial force due to prestressing. Previous research on clay masonry describes similar increase in masonry shear strength due to axial prestressing, e.g. Page and Huizer [5-6]. The first wall test, FG:L3.0-W20-P3, showed that the embedded horizontal shear reinforcement was not engaged at all. Therefore, sufficient shear strength from the masonry alone was assumed and the rest of the walls were constructed with no shear reinforcement. Five of the six fully grouted walls failed in flexure and only one wall failed in a brittle manner. It was unexpected that wall FG:L1.8-W15-P3 would fail due to tensile splitting in the diagonal compression struts, given the satisfactory performance of FG:L1.8-W15-P2. The lone diagonal crack that occurred in FG:L3.0-W15-P3 may indicate that this wall was close to failing in a similar manner.

Table 5.4 shows masonry shear strength predictions for the six FG walls according to NEHRP, Paulay and Priestley, and NZS 4230:1990 (shear reinforcement in W20 was not included in strength prediction), and the maximum wall strength recorded during the tests. It is seen from Table 5.4 that NZS 4230:1990 underestimates the actual masonry shear strength in all cases, despite the failure mode. The Paulay and Priestley prediction appears to underestimate the shear strength of walls FG:L3.0-W20-P3 and FG:L3.0-W15-P3, while providing an upper bound for the rest of the FG walls. It is interesting that the Paulay and Priestley equation accu-

<table>
<thead>
<tr>
<th>Wall</th>
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<th>Paulay and Priestley</th>
<th>NZS 4230:1990</th>
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<td>$V_{nehp}$</td>
<td>$V_{pp}$</td>
<td>$V_{mzs}$</td>
</tr>
<tr>
<td>1. FG:L3.0-W20-P3</td>
<td>0.984*</td>
<td>0.994</td>
<td>0.763</td>
<td>0.576</td>
</tr>
<tr>
<td>2. FG:L3.0-W15-P3</td>
<td>1.107*</td>
<td>1.199</td>
<td>0.947</td>
<td>0.576</td>
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<td>3. FG:L3.0-W15-P2C</td>
<td>0.888*</td>
<td>1.290</td>
<td>1.000</td>
<td>0.576</td>
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<tr>
<td>4. FG:L3.0-W15-P2E</td>
<td>0.888*</td>
<td>1.227</td>
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<tr>
<td>5. FG:L1.8-W15-P2</td>
<td>0.706*</td>
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<td>0.545</td>
</tr>
<tr>
<td>8. UG:L1.8-W10-P2</td>
<td>0.928$^+$</td>
<td>1.278</td>
<td>1.040</td>
<td>0.576</td>
</tr>
</tbody>
</table>

*flexural failure; $^+$shear failure; $^+$compression strut failure
rately predicts the strength at which FG:L1.8-W15-P3 failed. The NEHRP expression does in all cases predict shear strengths higher than the maximum measured strengths.

It is noted that the geometry of the utilised concrete masonry units is likely to significantly influence the masonry shear strength. For example, one can compare a wall constructed entirely from masonry ‘open end’ units, resulting in horizontal discontinuity of the grout core, and a wall constructed with H-units that allow both vertical and horizontal grout continuity. It is clear that the latter solution provides a more monolithic wall, being likely to possess higher masonry shear strength. Note however that no testing was conducted specifically to validate this conclusion.

5.5.1.6 Sliding propensity

Wall sliding was generally not a problem. However, for wall FG:L3.0-W15-P2E significant sliding occurred. This phenomenon may (and should) be eliminated by ensuring adequate shear friction between wall and foundation by requiring a certain magnitude of roughness of the wall to foundation interface, refer to Chapter 8 for an in-depth discussion. Alternatively, the wall could be recessed for example 100 mm into the foundation beam, or dowel bars could be incorporated in the construction joint to arrest any sliding.

5.5.1.7 Axial load ratio

The initial axial load ratio in the wall highly affects the wall performance. Realistic ratios are found in the interval between, say, 0.05 and 0.20. Self-weight constitutes a significant component of the axial load ratio. In a multi-storey masonry building, self-weight of the above floors may cause masonry stress at the foundation level of 1.0 MPa or more, i.e. an axial load ratio component of 0.06 for 18 MPa concrete masonry.

The lower bound is due to practical considerations in terms of prestressing losses (i.e. shrinkage, creep, etc.) and due to serviceability limit state considerations such as first cracking and lateral wall stiffness. The lowest ratio for this test series was 0.077 for wall FG:L3.0-W15-P2C, which performed satisfactorily.

The upper bound serves to keep the axial reinforcing ratio reasonably low. A high ratio also reduces the efficiency of the wall in terms of flexural strength, due to a relatively long compression zone, and reduces the wall displacement capacity. It is furthermore difficult to achieve satisfactory distribution of high prestressing anchorage forces in a concrete masonry wall, e.g.
Page and Shrive [5-7]. Wall FG:L1.8-W15-P3, that failed due to tensile splitting of diagonal compression struts, had a relatively high axial force ratio of 0.167. The concentration of pre-stressing tendons in the middle of that wall may have caused localised stress concentration in the top of the wall, potentially initiating the diagonal cracking.

5.5.2 Partially and ungrouted walls

5.5.2.1 Masonry shear strength

Prediction of shear strength of partially grouted and ungrouted walls follows the same principles as outlined for fully grouted walls, using the initial total tendon force, P. Evaluation of the wall width for the purpose of calculating shear strength for partially and ungrouted masonry should, however, be based on the sum of the masonry unit flange thicknesses. Singular grouted cells or masonry unit webs are not included in the shear flow through the panel and therefore cannot be expected to carry shear (Paulay and Priestley [5-4]). For walls PG:L3.0-W15-P2 and UG:L1.8-W10-P2, the effective wall thickness for shear calculation amounted to 63 mm and 61 mm respectively.

Partially grouted and ungrouted wall shear demand and masonry shear strength comparison is provided in Table 5.4. The shear demand is determined from the maximum shear carried by the walls before shear failure. Predicted masonry shear strength was calculated according to Eqns. 5.1 through 5.3 using a masonry crushing strength of 13.3 MPa for PG:L3.0-W15-P2 and 15.0 MPa for UG:L1.8-W10-P2. It is noted that the masonry crushing strength for PG:L3.0-W15-P2, given in Table 5.1, was specific to a grouted cell, while the above value constitutes an average for the partially grouted wall.

It is seen from Table 5.4 that the masonry shear strength predicted by NZS 4230:1990, Eqn. 5.3, gives conservative estimates of actual masonry shear strength. The prediction was reasonably close for the squat wall (L3.0) while rather low for the slender wall (L1.8). Comparison of test results with the masonry shear strengths predicted by the NEHRP provisions and that advocated by Paulay and Priestley, indicate that Eqns. 5.1 and 5.2, overestimate the actual shear strength considerably, and are therefore unconservative for partially and ungrouted walls.
Fig. 5.9—PG and UG Walls, F-D envelopes, normalised with $V_p$.

5.5.2.2 Flexural strength

Fig. 5.9 shows the force-displacement envelopes for PG:L3.0-W15-P2 and UG:L1.8-W10-P2, normalised with respect to $V_f$. It is observed for both walls that the measured wall strength exceeded $V_f$. This is of significant relevance when conducting non-ductile strength design, where nominal flexural strength must exceed the flexural demand, but there is no requirement for reliable inelastic response.

5.5.2.3 Vertical strain

The masonry vertical strain was evaluated at nominal flexural strength, which was achieved for both walls. Table 5.3 shows that strains of 0.0016 to 0.0018 were measured at the predicted nominal strength, $M_n$.

5.5.2.4 Axial load ratio

The axial load ratio for partially and ungrouted walls directly affects both the nominal flexural strength and the masonry shear strength, as suggested by Eqns. 8.17 and 5.1 to 5.3. These walls are not expected to perform in a ductile manner, therefore there is no need to provide reserve tendon strain capacity allowing non-linear elastic rocking behaviour. Consequently, the pre-stressing steel area should be determined from the desired serviceability limit state capacity or nominal flexural capacity using the maximum allowed tendon stress, taking due consideration for prestress losses.
The axial load ratio that partially and ungrouted walls can sustain is probably of the same magnitude as that suggested for fully grouted walls. It is however emphasised that proper diffusion of the concentrated force under the prestressing anchorages is critical.

5.6 CONCLUSIONS AND FUTURE RESEARCH

5.6.1 Fully grouted walls

It is concluded that fully grouted unbonded post-tensioned concrete masonry is a competent material combination for ductile structural wall systems. The PCM walls reported herein exhibited a nearly non-linear elastic behaviour dominated by rocking response. Large drift capacity of more than 1.4% was measured and still larger drift capacity can be expected from higher walls.

Only localised damage occurred, as shown in Fig. 5.3, making earthquake damage easy to repair. Possible tendon force loss could be compensated for by re-stressing of the tendons. This appears feasible because tendon strains are expected to stay below 1% for typical wall dimensions and realistic drift ratio demands when tendon yielding occurs over the entire unbonded length.

Tendon force loss due to yielding should, for short (squat) walls, be expected at relatively low wall drift ratios. Even after tendon yielding, reliable and self-centring wall behaviour is expected.

Measurements at nominal flexural strength, \( M_n \), suggest an extreme masonry fibre strain of the order of 0.003. Strains as high as 0.007 may be considered for design if the true ultimate condition is assumed as being the stage at which toe distress initiates.

Clearly, the additional axial load from the prestressing enhanced the concrete masonry shear strength. It was concluded that fully grouted PCM walls of dimensions similar to those described above, in most cases did not need horizontal shear reinforcement in order to develop flexural/rocking response. However, one fully grouted wall with high axial prestressing force failed by tensile splitting of diagonal compression struts, suggesting that this issue should be pursued further.

The Paulay and Priestley [5-4] recommendation for masonry shear strength accurately predicted the strength of FG:L1.8-W15-P3, however this failure was attributed not attributed
directly to shear but to tensile splitting of diagonal compression struts. The NZS4230:1990 shear strength provision is highly conservative and seems to unsatisfactorily reflect the large increase in masonry shear strength due to the axial prestressing force.

Relatively little energy dissipation was observed during wall cycling. This may impact the use of such walls for ductile seismic design in the sense that additional damping may be required to control the structural drift demand in an earthquake event. Further research is needed to investigate the relationship between PCM wall strength and drift demand arising from seismic excitation.

5.6.2 Partially and ungrouted walls

Testing clearly showed that partially and ungrouted PCM walls were capable of developing significant strength, exceeding the predicted nominal flexural strength. This suggests that partially and ungrouted PCM walls may suitably be designed using strength design.

The wall tests initially displayed flexural behaviour but ultimately developed shear response. Diagonal shear cracking was the failure mode. Despite this brittle failure mode, significant displacement capacity was recorded. These walls did not develop reliable non-linear elastic behaviour.

The additional axial force due to prestressing clearly enhanced the concrete masonry shear strength. Test results suggest that none of the shear strength equations performed well. It is, however, safe to use the NZS 4230:1990 code requirements when predicting concrete masonry shear strength for ungrouted and partially grouted walls, basing masonry shear strength on the concrete masonry unit flange thickness.
5.7 REFERENCES


STRUCTURAL TESTING - SERIES 2

6. ENHANCED SINGLE-STOREY PCM WALLS

In-plane response of post-tensioned concrete masonry walls with unbonded tendons, incorporating strengthened masonry and enhanced energy dissipation, is examined by means of structural testing. An introduction to the Series 2 wall testing programme is followed by a presentation of the results from structural testing of the five fully grouted walls. Discussion of the results is concerned with wall structural response in terms of flexural strength, displacement capacity, tendon stress and masonry vertical strain, and makes comparison with Series 1 testing.

6.1 INTRODUCTION

Previous testing presented in Chapter 5 explored post-tensioned concrete masonry (PCM) walls made with un-confined masonry. It was found that such walls typically had reliable displacement capacity (drift capacity) of up to 1%, however, in some cases the limited strain capacity of un-confined masonry resulted in masonry crushing and subsequent strength degradation, and prohibited development of further reliable drift capacity.

One of the principal intents of this testing series was to investigate means of masonry strengthening that were expected to allow for reliable drift capacity, say of the order of 1.5%. Drift demands of such magnitude are likely to be encountered in structural systems utilising unbonded post-tensioned walls when subjected to seismic loading because of the inherent limited structural damping, see Chapter 9.

As a consequence of limited hysteretic energy dissipation, additional structural damping may be required to control the wall displacement demand during seismic activity. As a potentially simple solution, ‘dog-bone’-type dampers were incorporated in one wall test to explore the efficiency of such a device.
Two of the tested walls incorporated a hydraulic system for application of constant axial load in the prestressing bars, thus entirely avoiding prestress loss due to tendon yielding.

The single-storey wall testing results reported herein enabled further development and verification of structural behaviour prediction methods for prestressed concrete masonry walls which are currently being developed at the University of Auckland. Recommendations from this testing phase, having explored strengthening and energy dissipation devices, are incorporated in the design and testing of the two large scale multi-storey walls, as reported in Chapter 7. These 2/3-scale walls represented a realistic 4-5 storey masonry prototype structure. In that respect, testing Series 1 and 2 may be regarded as component testing enabling the author to select and design an appropriate structural configuration to be used for the large scale testing.

Detailed descriptions of materials, wall construction, test setup, instrumentation and testing procedure, pertaining to similar testing, may be found in Chapter 5.

6.2 CONSTRUCTION DETAILS

6.2.1 Wall specifications

Wall dimensions are specified in Table 6.1. The wall designations follow the convention presented in the previous chapter. Further additions are: CP, HB, ED and CA, which represent confining plates, high strength blocks, energy dissipation and constant axial load, respectively. In the following discussion of Series 2 walls, the designations FG:L3.0-W15 have been left out for convenience, however, when comparing with Series 1 results the proper wall designations have been resumed.

\[ f_m = (P+N) / (l_w b_w) \]

**TABLE 6.1—Wall specifications**

<table>
<thead>
<tr>
<th>Wall</th>
<th>length</th>
<th>thickness</th>
<th>self weight</th>
<th>masonry strength</th>
<th>tendons</th>
<th>initial steel stress</th>
<th>initial presstr. force</th>
<th>axial force ratio^a</th>
</tr>
</thead>
<tbody>
<tr>
<td>FG:L3.0-W15-P1-CP</td>
<td>3.0</td>
<td>140</td>
<td>41</td>
<td>18.2</td>
<td>1</td>
<td>720</td>
<td>299</td>
<td>0.044</td>
</tr>
<tr>
<td>FG:L3.0-W15-P2-CP</td>
<td>3.0</td>
<td>140</td>
<td>41</td>
<td>15.1</td>
<td>2</td>
<td>703</td>
<td>584</td>
<td>0.099</td>
</tr>
<tr>
<td>FG:L3.0-W15-P2-CP-CA</td>
<td>3.0</td>
<td>140</td>
<td>41</td>
<td>20.6</td>
<td>2</td>
<td>708</td>
<td>588</td>
<td>0.073</td>
</tr>
<tr>
<td>FG:L3.0-W15-P2-CP-CA-ED</td>
<td>3.0</td>
<td>140</td>
<td>41</td>
<td>17.8</td>
<td>2</td>
<td>727</td>
<td>604</td>
<td>0.086</td>
</tr>
<tr>
<td>FG:L3.0-W15-P2-HB</td>
<td>3.0</td>
<td>140</td>
<td>41</td>
<td>12.5</td>
<td>2</td>
<td>743</td>
<td>617</td>
<td>0.125</td>
</tr>
</tbody>
</table>

^aunconfined strength, full height units, \( f_m = (P+N) / (l_w b_w) \)
All test units were constructed to the same dimensions, a length, \( l_w \), of 3.0 m, a height, \( h_w \), of 2.6 m and a thickness, \( b_w \), of 0.14 m using 15 series concrete masonry units. The actual thickness of the walls was approximately 10 mm less than the nominal masonry unit thickness of 150 mm. The main part of each wall was constructed with full height concrete masonry units (nominally 200 mm high). One lower corner of all P2-CP walls (corner in compression under push loading) was constructed with half-height concrete masonry units (100 mm nominal height), as indicated in Fig. 6.1, with an extend of 600 mm vertically and 1200 mm horizontally. The asymmetrical use of half height units in the bottom corners enabled a direct comparison between the effectiveness of different confining plate details. The half height units were obtained from the masonry block supplier, though some webs needed to be modified to accommodate the shear reinforcement. Both lower corner units of the HB wall were fabricated specially for this test using high strength steel fibre reinforced concrete, as shown in Fig. 6.2.

Confining plates embedded in the toe regions (flexural compression zones) are indicated on Fig. 6.1. These plates were placed in all bed joints in the lowest 600 mm above the foundation, embedded with mortar on both sides for best integrity. In the following U200, CP200 and CP100 refer to 'unconfined full height grouted concrete masonry' (main body of walls), 'con-
fined full height grouted concrete masonry’ (confined region bottom left corner in Fig. 6.1) and ‘confined half height grouted concrete masonry’ (confined region bottom right corner in Fig. 6.1), respectively. Further information on enhancement of the masonry performance caused by confining plates may be found in Chapter 3.

High strength corner units were manufactured to the exact dimension of ordinary 15 Series concrete masonry units. Using these units, it was attempted to provide durable pivoting points for wall rocking, thus isolating the above units from extreme strain. The high strength steel fibre concrete measured an average cube strength of 131 MPa, further details below.

Walls P2-CP-CA and P2-CP-CA-ED had 2 x HD10 (10 mm bar, 430 MPa nominal yield strength) embedded horizontally at 400 mm vertical intervals. No shear reinforcing was provided in the other walls (except for 2 x HD10 placed in top course to provide a bond beam).

Energy dissipators were embedded centrally in P2-CP-CA-ED as shown in Fig. 6.1. These were of the so called ‘dog bone’-type, and were expected to provide hysteretic damping as a result of yielding in tension and compression. The milled part of the bars were designed such that the axial prestressing force (kept constant throughout testing) comfortably could yield the bars in compression, thus forcing the wall back to its original alignment upon unloading. An
energy dissipator yield force to prestressing force ratio of approximately 1:3 was chosen for this test, reflected by nominal yield strength of the reduced diameter section of each energy dissipation bar of approximately 100 kN and axial force in the wall due to the prestressing of 617 kN. Fig. 6.3 illustrates the dimensions of the energy dissipation bars. The bars were made of 20 mm reinforcing steel with nominal yield strength of $f_y = 500$ MPa and ultimate strength ranging between $1.15f_y$ and $1.4f_y$. The bar diameter was reduced to 16 mm over a length of 375 mm, to ensure that the bar yielded in a well defined region adjacent to the wall base, and to limit strain such that an extreme wall displacement excursion would not cause bar rupture. The bar was confined by heavy steel tube with an inside diameter of 27 mm and a length of 555 mm to ensure that the section of bar intended to yield would not buckle when put into compression. The tube was filled with dental plaster and the dissipation bars remained unbonded over a length of 465 mm, as shown in Fig. 6.3.

All walls were prestressed with two 23 mm VSL thread bars placed concentrically at ±400 mm from the wall centreline. The prestressing tendons remained unbonded over the entire wall height and had a typical unbonded length of 3.5 m, including unbonded lengths in the foundation (300 mm), loading beam (400 mm) and load cells on top of the loading beam (200 mm). An additional tendon length of 300 mm due to hydraulic jacks in the vertical loading system applied to the CA walls. The wall initial prestressing force and axial force ratios (due to prestressing and self-weight) are given in Table 6.1. Unfortunately, the left prestressing bars (bar closest to strong wall) in test unit P1-CP tore out of the foundation anchorage during the initial stressing, thus the designation ‘P1’. The damage could not be repaired with the wall in the test-
As a result an alternative testing procedure was instated for this wall, consisting of testing the wall in the pull direction only to maximise the prestressing bar eccentricity.

Constant axial load in the prestressing bars was applied for the walls CP-CA and CP-CA-ED. For these particular walls, the prestressing force was kept constant, such that the bars did not yield as the wall was displaced laterally. Two identical hydraulic jacks were connected in parallel to the same pump and remained in place throughout the test. This ensured that the force applied in each bar was kept the same and constant as the wall was rocked back and forth.

6.2.2 Wall construction

Prior to wall construction, the prestressing bars were inserted into the foundation and encased by ducting to ensure proper isolation from the grout. The walls were constructed in running bond by experienced blocklayers under supervision, using standard grey precast concrete masonry units and DRICON™ trade mortar. Open-ended concrete masonry units were used to avoid having to thread the masonry units onto the 4 m long prestressing bars. Grouting with 17.5 MPa ready-mixed regular block fill, containing SIKA CAVEX™ expansion agent, was generally performed the following day. Vibration of the grout was carried out for the CA walls.

6.2.3 Material properties

Average masonry material properties were determined by material testing, typically in samples of three. The masonry crushing strength $f'_m$, elastic modulus $E_m$, grout crushing strength $f'_g$ (200 mm high, 100 mm diam. cylinder), and average age at day of testing are given in Table 6.2.

High strength masonry units were fabricated with steel fibre reinforced concrete of the following mix ratio: 1010 kg of dry silica sand, 705 kg of ordinary portland cement, 230 kg Microsil-

### Table 6.2—Material properties

<table>
<thead>
<tr>
<th>Wall</th>
<th>$f'_m$</th>
<th>$E_m$</th>
<th>$f'_g$</th>
<th>Age at DOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>FG:L3.0-W15-P1-CP</td>
<td>18.2</td>
<td>18.6</td>
<td>15.0</td>
<td>13</td>
</tr>
<tr>
<td>FG:L3.0-W15-P2-CP</td>
<td>15.1</td>
<td>16.2</td>
<td></td>
<td>63</td>
</tr>
<tr>
<td>FG:L3.0-W15-P2-CP-CA</td>
<td>20.6</td>
<td>19.8</td>
<td>15.0</td>
<td>38</td>
</tr>
<tr>
<td>FG:L3.0-W15-P2-CP-CA-ED</td>
<td>17.8</td>
<td>14.9</td>
<td>14.3</td>
<td>31</td>
</tr>
<tr>
<td>FG:L3.0-W15-P2-HB</td>
<td>12.5*</td>
<td></td>
<td></td>
<td>19.0</td>
</tr>
</tbody>
</table>

* CAVEX not applied

- 116 -
ica 600, 210 kg of crushed silica sand, 16 kg of super plasticiser (Visocrete-5, SIKA NZ, Ltd.), 200 kg of water and 60 kg of Fortex steel fibres produced by Novocon International (dosage rate of approximately 2.5% by weight). The mix resulted in an average 50 mm cube crushing strength of 131 MPa and a standard deviation of 2 MPa.

VSL 23 mm diameter high strength threaded bars were used in all wall tests and had the typical properties: yield strength of 970 MPa, ultimate strength of 1160 MPa, ultimate strain of 8% and modulus of elasticity of 190 GPa, all of which were confirmed by material testing.

6.3 TESTING DETAILS

6.3.1 Test setup
Wall testing reported in this chapter was conducted using the setup discussed in Chapter 5 and shown in Fig. 5.1(a), consisting of a reusable foundation and loading beam. Notably, no effort was made to replicate gravity and live loads from suspended floors, with the assumption that these loads were directly analogous to a component of the applied post-tensioning force.

Fig. 6.4 schematically shows typical wall instrumentation. Lateral load was measured by two load cells positioned in series with the lateral hydraulic actuators, LCH1 and LCH2, and lateral displacement at the top of the wall (height of 2.8 m above base) was measured by the displacement transducers DISP1 and DISP2. Wall flexural deformation and vertical strain were measured by instruments denoted ‘F’. Relative sliding displacements between loading beam/wall, wall/foundation and foundation/strong floor were measured by instruments denoted ‘S’. ‘DIGI’ denotes digital display displacement transducers which were employed to confirm lateral displacement measurements. Axial force in the prestressing bars was measured by the load cells LCPR1 and LCPR2. Flexural instrumentation F13-F24, shown in Fig. 6.4(a), was mounted on one side only, whereas instruments F1-F4, F9-F12 and F17-F28 shown in Fig. 6.4(b) were mirrored on the wall back side.

Dowel bars were embedded in the foundation for wall tests P2-CP-CA and P2-CP-CA-ED, as indicated in Fig. 6.1. The dowels were made from HD20 reinforcing steel, had an embedment length into the foundation of 200 mm and protruded 80 mm into the wall. The deformations were removed from the 80 mm length and the bar was greased and encased in a sleeve to ensure that no bond was formed between the dowel and surrounding grout.
Sliding blocking devices were mounted on the foundation beam by each end of the wall. These devices were intended to block wall sliding, should this occur. The height of the devices varied from 70 mm for walls P1-CP, P2-CP, P2-CP-CA and HB, to 150 mm for P2-CP-CA-ED. Gaps
of 3 mm (nominal) were present between the wall ends and the sliding blocking devices in the unloaded state.

6.3.2 Testing procedure

Cyclic structural testing was conducted according to the procedure outlined by Park [6-1], with the ductility one displacement here termed 'displacement level one' or D1. After definition of D1, the walls were cycled to D2, D4, D6, etc. until failure, with Dn being defined as n times D1. Prior to executing this loading procedure, serviceability limit state level force cycles were applied to some walls in order to investigate the uncracked stiffness, the lateral displacement profile and the initial crack formation.

6.3.3 Predicted nominal flexural strength

For a symmetric distribution of post-tensioning and gravity actions, the nominal flexural strength, \( M_n \), is defined by the New Zealand Masonry code NZS 4230:1990 [6-2] as given in Eqns. 8.17, 8.18 and 8.29. Details on these equations can be found in Chapter 8. The base shear, \( V_b \), corresponding to \( M_n \) may be calculated using Eqn. 8.17, where \( P \) represents the initial prestress force (no lateral load applied to wall), \( \Delta P \) is the total tendon force increase due to wall deformation at \( V_b \), \( l_w \) is the wall length, \( h_t \) the height of the wall and \( N \) is the axial load at the base of the wall due to live and dead load. The equivalent rectangular compression zone length, \( a \), is defined by Eqn. 8.18 as a function of the axial forces on the section, the masonry compressive strength, \( f'_{m} \), the wall width \( b_w \) and \( \alpha = 0.96 \) (confined grouted concrete masonry). Eqn. 8.29 defines the effective tendon force eccentricity, \( e_n \), with respect to the initial prestressing force and the tendon force increase based on the assumption of uniformly distributed prestressing steel. In the absence of suitable procedures for estimating \( \Delta P \) at the time of testing, the tendon force increase at nominal strength was estimated to be 50 kN for walls P2-CP and P2-HB and zero for P1-CP. Walls P2-CP-CA and P2-CP-CA-ED had constant axial load applied to the prestressing bars, i.e. \( \Delta P = 0 \).

Flexural strength increase of wall P2-CP-CA-ED, due to the yielding energy dissipators, was taken into account when estimating \( V_f \) by assuming a steel stress of \( f_y = 500 \text{ MPa} \).
6.3.4 Predicted masonry shear strength

The predicted masonry shear strength, $V_m$ (force), was calculated according to Eqn. 5.2 as advocated by Paulay and Priestley [6-3]. Shear strength due to shear reinforcing was calculated using the equation:

$$V_{st} = A_s f_{vy} \frac{d}{s}$$  \hspace{1cm} (6.1)

where $A_s$ is the steel area with centre to centre spacing $s$, $d = 0.8 l_w$ and $f_{vy}$ is the nominal yield strength of the shear reinforcing. The predicted wall shear strengths $V_s = V_m + V_{st}$ are listed in Table 6.3. The initial prestress force was used in this calculation.

6.4 TEST RESULTS

This section describes the behaviour of the five walls and presents the measured masonry compression toe strains at various wall states. General wall behaviour is summarised in Table 6.3, where $V_{max}$ is the maximum lateral force recorded and $d_{vmax}$ is the corresponding displacement. The ultimate displacement capacity, $d_u$, is defined as the point at which the lateral wall strength had degraded below 80% of $V_{max}$. The ultimate drift capacity is defined as $\gamma_u = d_u/h_c$ with $h_c = 2.8$ m.

Rocking response was recorded for all wall tests, with a single large crack opening up along the wall-foundation construction joint. No distributed flexural cracks were observed. A singular inclined crack was observed for wall P2-CP as a result of loading in the pull direction. A similar crack was observed for wall P2-HB. Both of these walls contained no shear reinforcing. Cracking occurred in the centre of wall CP-CA-ED, due to bond stress between the energy dissipation bars and the masonry. High initial stiffness was recorded. At nominal flexural

<table>
<thead>
<tr>
<th>Wall</th>
<th>Prediction</th>
<th>Results</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. FG:L3.0-W15-P1-CP</td>
<td>216 325</td>
<td>Pull/CP200</td>
<td>Masonry crushing, flexural failure, pull test only</td>
</tr>
<tr>
<td>2. FG:L3.0-W15-P2-CP</td>
<td>333 372</td>
<td>Push/CP100, Pull/CP200</td>
<td>Masonry crushing, diagonal crack, flexural failure</td>
</tr>
<tr>
<td>3. FG:L3.0-W15-P2-CP-CA</td>
<td>311 815</td>
<td>Push/CP100, Pull/CP200</td>
<td>Masonry crushing, flexural failure</td>
</tr>
<tr>
<td>4. FG:L3.0-W15-P2-CP-CA-ED</td>
<td>400 842</td>
<td>Push/CP100, Pull/CP200</td>
<td>Masonry crushing, flexural failure</td>
</tr>
<tr>
<td>5. FG:L3.0-W15-P2-HB</td>
<td>339 360</td>
<td>Push/HB</td>
<td>Masonry crushing, diagonal crack, flexural failure</td>
</tr>
</tbody>
</table>

Note: sliding displacement subtracted from $d_{vmax}$ and $d_u$
strength, \( V_f = \frac{M_v}{h_w} \), wall softening occurred due to the initiation of rocking. Tendon yielding was experienced for walls P1-CP, P2-CP and P2-HB, which all had tendons permanently anchored in the foundation and loading beam. The tendons in walls P2-CP-CA and P2-CP-CA-ED remained elastic during testing.

The failure mode was characterised by localised masonry crushing in the compression toe regions, resulting in gradual strength degradation. Using CP200 masonry, inelastic masonry strain were found to be concentrated in the lowest course (200 mm), whereas when using CP100 masonry inelastic vertical strain penetrated into the second and third course above the foundation (200-300 mm). Failure of the compression toes for wall P2-HB was initiated by crushing of the ordinary masonry course (U200) just above the high strength units.

Considerable sliding displacement between wall and foundation was recorded for P1-CP, P2-CP and P2-CP-CA.

Hysteretic energy dissipation was increased considerably for P2-CP-CA-ED because of the embedded ‘dog-bone’ energy dissipation bars, in comparison with the other walls.

It is pointed out for P1-CP that, while results have been included in this chapter, relatively little attention has been given to this wall due to its unrealistic dimensions and prestressing configuration.

6.4.1 Force-displacement response

Fig. 6.5, depicting the force-displacement response for all fully grouted walls, demonstrates the nearly non-linear elastic behaviour exhibited by the tested unbonded post-tensioned walls prior to toe crushing, with the exception of P2-CP-CA-ED. It is seen that all walls returned to their original alignment, even after large displacement excursions. This type of response may be categorised as ‘origin oriented’. For all walls, except P2-CP-CA-ED, the individual curves for each excursion appear pinched, implying little hysteretic energy dissipation. It is noted that the term ‘elastic’ in a rigorous sense indicates reversibility and no accumulation of damage. In reality the walls did accumulate damage from the onset of tendon yielding and masonry crushing, causing a reduction in wall stiffness. Thus the behaviour was termed ‘nearly’ non-linear elastic.
None of the walls exhibited symmetrical response in the push and pull directions. P1-CP was tested in the pull direction only. P2-CP was tested cyclically (alternating push and pull excursions) for displacements up to D2 (5 mm) and thereafter cyclically, first in the push direction and then in the pull direction, because of an inclined crack (Fig. 6.6(b)) that occurred while pulling to -D2 (-5 mm) which caused concern about the wall integrity in the pull direction.
Continued testing in the push direction was adopted to at least capture large displacement response in one direction. All P2-CP walls had asymmetric strengthening in the wall corners as indicated in Fig. 6.1, thus different performance in push and pull directions. P2-HB had a symmetric layout as indicated in Fig. 6.2, nevertheless, its performance in the directions of loading differed. The asymmetric behaviour of P2-CP-CA, as seen in Fig. 6.5(c), was partially caused by extensive sliding in the push direction. Note that (1) all lateral displacement data depicted in Fig. 6.5 was corrected for sliding displacement, thus representing only the wall flexural and shear deformation behaviour and (2) the corrected lateral displacement has been used as reference in all graphs and tables in this chapter. Sliding displacement data is presented in section 6.4.3.

6.4.2 Damage pattern and failure mode

Gradual strength degradation was observed for all walls (excepting P2-HB), attributed to spalling of face shells and crushing of the grout core. Large displacement capacity was observed with reliable wall resistance for drifts, $\gamma$, ranging from 0.7% to more than 2.6%, refer to Table 6.3. A relatively more sudden strength degradation was observed for P2-HB in the push direction when going beyond D12 (30 mm), caused by rapid strength degradation of the ordinary masonry block (U200) just above the high strength block (lower right wall corner as shown in Fig. 6.6(e)). Crack patterns after failure, depicted schematically in Fig. 6.6, indicated that the extent of damage was confined to the lowest two masonry courses in the toe regions, with the majority of damage concentrated in the lowest course. Damage to the masonry units immediately above the lowest course generally was limited to vertical splitting cracking near the wall end. The inclined cracking observed in P2-CP (no shear reinforcing) and P2-HB (no shear reinforcing) suggests that shear reinforcement is necessary for walls of present dimensions, in order to limit sporadic cracking in the diagonal compression strut. Cracking observed in P2-CP-CA-ED (with shear reinforcing) was mainly due to localised stress concentration arising from mobilisation of the energy dissipator strength.

Figs. 6.6(b) and (e) showing inclined cracks that appeared while pulling P2-CP and P2-HB to -D2 (-5 mm) and -D4 (-10 mm), respectively. These crack are believed to have been caused by large localised splitting forces associated with the prestress anchorage at the centre of the loading beam. The cracks did not develop further during testing, possibly due to transverse restraint provided by the loading beam.
Fig. 6.6—Damage accumulation at failure
Fig. 6.7—Relative sliding between wall and base

6.4.3 Sliding

Fig. 6.7 presents the recorded sliding displacement for the five wall tests; again it is noted that lateral displacement on the horizontal axis has been corrected for sliding displacement. It appears from Figs. 6.7(a) and (b) that significant sliding occurred for P1-CP and P2-CP...
throughout the tests, and that sliding increased gradually with lateral displacement. Both tests were conducted in one direction only at large displacement levels which naturally did tend to increase the total sliding displacement due to the absence of excursions in the opposite loading direction that could reverse the sliding that occurred. Sliding displacements were negligible for P2-CP-CA for all cyclic excursions, see Fig. 6.7(c). Once the wall had failed in the pull direction (CP200) and the testing in the pull direction was terminated at -D10 (-30 mm), the wall slid further and further in the push direction. This was partially due to reduced efficiency of the dowel bars as the compression zone integrity deteriorated. There was no indication of significant wall sliding relative to the foundation for P2-CP-CA-ED as indicated in Fig. 6.7(d), attesting to the efficiency of the dowel and energy dissipator bars to block sliding. Fig. 6.7(e) shows that negligible sliding occurred for P2-HB until after D16 (40 mm), where push testing was terminated. Still sliding displacement was relatively small.

Gradual deterioration of the foundation beam roughness had occurred as the testing series progressed (foundation beam also used for Series 1 testing), thus reducing the shear friction along the wall/base construction joint.

6.4.4 Prestressing force

The prestressing tendon force histories (sum of force of all tendons) for all walls are plotted against the lateral displacement in Fig. 6.8. For reference, it is noted that the nominal yield strength for one 23 mm VSL CT stress bar is approximately 385 kN.

Walls P1-CP, P2-CP and P2-HB had the tendons rigidly anchored in the foundation and on top of the loading beam, and did exhibit tendon yielding during testing, as indicated in Figs. 6.8(a), (b) and (e). It is seen that the first significant total tendon force loss (more than 5% of the initial force) for these walls occurred after lateral wall displacements to about ±10 mm, in all cases roughly corresponding to first yield of the extreme tendons when considering the wall as a rigid body rocking about the wall corners. At the conclusion of testing of walls P1-CP, P2-CP and P2-HB, between 20% and 50% of the initial total prestressing force remained.

It is noted in Fig. 6.8(e) that the total prestress loss never approached the nominal yield strength of the two tendons of approximately 770 kN despite recording prestress loss. This was due to uneven force in the tendons due to flexural action and inaccuracy in the tendon force measurement.
Fig. 6.8—Prestressing force histories

Figs. 6.8(c) and (d) show the prestressing tendon force history for P2-CP-CA and P2-CP-CA-ED. These figures reveal that the prestressing force at the excursion displacement peaks amounted to approximately 610 kN and 640 kN for P2-CP-CA and P2-CP-CA-ED, respectively, or about 5% higher than the initial prestress force value. It was concluded that the hydraulic system controlled the prestressing force with satisfactory accuracy.
6.4.5 Vertical masonry strain

Vertical strain recorded along the bottom of the walls during testing is shown in Figs. 6.9 through 6.13. Each figure shows strain plots for instrument levels 1 and 2 above the base, i.e. average strain for 0-100 mm and 100-300 mm above the foundation for P1-CP and P2-HB, and average strain for 0-200 mm and 200-400 mm above the foundation for the other walls. For walls with flexural instrumentation on both sides, the average strain of both sides was used. The sign convention in these figures defines compression strain as negative. The vertical dashed lines in the figures indicate the extremities of walls, i.e. the extreme masonry fibres at ±1.5 m from the wall centre line. Figures (a) and (c) relate to low displacement response, all excursions up to and including displacement level D2, and (b) and (d) relate to response beyond D2.

Regarding vertical strain measurements for P1-CP and P2-HB, it is suggested in Fig. 6.4(a) that the instruments F6, F16, F20, F24 and F12 were spanning between the foundation beam and the middle of the first masonry course. Likewise, instruments F5, F15, F19, F23 and F11 spanned between the middle of first and second courses. This implies that both level 1 and level 2 vertical strain measurements for these walls included a component of vertical deformation of the first course, the course where most inelastic action occurred. Strain measurements for all other walls were recorded between bed joints of one course for HB and CP200 configurations and between bed joints of two courses for CP100.

No plots of vertical strain above level 2 have been included in this report because the recorded vertical strain indicated elastic masonry response at these locations (vertical strain significantly lower than 0.001). This attests that the vertical extent of the compression zone undergoing plastic deformation remained rather short, of the order of 200 mm to 400 mm. Furthermore, the resolution of the utilised instruments of perhaps 0.0005 strain excluded meaningful interpretation of data recorded above level 3. It is acknowledged that vertical strain readings at level 1 were influenced by base sliding to some degree. It is remarked that the data plotted in Figs. 6.9 through 6.13 are associated with some degree of uncertainty, especially for displacement levels above D2 where significant deterioration of the masonry units may have affected instrument reading.
Fig. 6.9—FG:L3.0-W15-P1-CP, Horizontal strain profiles, pull direction only
Fig. 6.10—FG:L3.0-W15-P2-CP, Horizontal strain profiles
Fig. 6.11—FG:L3.0-W15-P2-CP-CA, Horizontal strain profiles
Fig. 6.12—FG113.0-W15-P2-CP-CA-ED, Horizontal strain profiles
Fig. 6.13—FG:L3.0-W15-P2-HB, Horizontal strain profiles
Figs. 6.9(a)+(b) through 6.12(a)+(b) reveal that vertical strain varied nearly linearly along the length of the walls (except P2-HB) for all displacement levels. Figs. 6.13(a)+(b) show that this was not the case for P2-HB for loading above ±Vₚ, presumably because of the stiffness incompatibility between high strength and ordinary masonry units.

The general trend for CP200 and CP100 masonry for low level response (Figs. (a)) were recorded extreme fibre strain of approximately 0.003 for readings below D2 and approximately 0.004-0.006 for D2. The extreme fibre strain for large displacement response ranged from 0.01 to 0.08. Strains at level 2 remained below 0.0012 for P2-CP, P2-CP-CA, and P2-CP-CA-ED, thus indicating elastically responding masonry. Measurements at level 2 for P1-CP and P2-HB suggest strains higher than 0.0015, however, as noted above, these measurements were taken by instrumentation that also picked up deformation in the first masonry course.

Table 6.4 presents a summary of vertical strain measurements related to specific events occurring during testing. Vertical strains in the flexural compression zones (the toe) of the walls were evaluated from data acquired during testing for the following wall states: (1) at nominal flexural strength, Vₚ, (2) at maximum strength, Vₚmax, and (3) at ultimate displacement, dᵤ. Refer to Table 6.3 for Vₚ, Vₚmax and dᵤ. The measured strains in the extreme compression fibre at the wall states described above are listed in Table 6.4 for the push and pull directions, as available. The strains were measured in the lowest 100 to 200 mm of the walls. Results for P2-HB are not included.

### Table 6.4—Measured masonry vertical strain

<table>
<thead>
<tr>
<th>Wall</th>
<th>Dir./Conf.</th>
<th>Vₚ displ.</th>
<th>Vₚ strain</th>
<th>Vₚmax strain</th>
<th>dᵤ strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. FG:L3.0-W15-P1-CP</td>
<td>Pull/CP200</td>
<td>-1.7</td>
<td>0.0035</td>
<td>0.0107</td>
<td>0.070*</td>
</tr>
<tr>
<td>2. FG:L3.0-W15-P2-CP</td>
<td>Push/CP100</td>
<td>1.7</td>
<td>0.0016</td>
<td>0.0230</td>
<td>0.036</td>
</tr>
<tr>
<td></td>
<td>Pull/CP200</td>
<td>-2.0</td>
<td>0.0019</td>
<td>0.0190</td>
<td>0.055</td>
</tr>
<tr>
<td>3. FG:L3.0-W15-P2-CP-CA</td>
<td>Push/CP100</td>
<td>4.0</td>
<td>0.0016</td>
<td>0.0037</td>
<td>0.068</td>
</tr>
<tr>
<td></td>
<td>Pull/CP200</td>
<td>-4.0</td>
<td>0.0029</td>
<td>0.0052</td>
<td>0.024</td>
</tr>
<tr>
<td>4. FG:L3.0-W15-P2-CP-CA-ED</td>
<td>Push/CP100</td>
<td>6.7</td>
<td>0.0027</td>
<td>0.0053</td>
<td>0.033</td>
</tr>
<tr>
<td></td>
<td>Pull/CP200</td>
<td>7.0</td>
<td>0.0055</td>
<td>0.0116</td>
<td>0.050</td>
</tr>
</tbody>
</table>

Note: *extrapolation based on instruments F16 and F20
Fig. 6.14—F-D envelopes, P2-CP, P2-CP-CA and P2-HB

6.5 DISCUSSION

6.5.1 Flexural response

Fig. 6.5 indicates that the developed flexural strength $V_{\text{max}}$ (base shear corresponding to the maximum moment) exceeded the predicted flexural strength $V_f$ in all wall test. Comparing the force-displacement envelopes (peak response of first excursion cycle for each displacement level) for walls P2-CP, P2-CP-CA and P2-HB, which had similar dimensions and prestressing properties, shown in Fig. 6.14, it is seen that P2-CP and P2-HB developed higher strength than P2-CP-CA. This directly resulted from the tendon force in P2-CP-CA being kept constant during testing, prohibiting tendon force increase due to flexural action. The performance of P2-CP and P2-CP-CA in the push direction (positive displacement), both featuring CP100 masonry in the compression zone, differed. It appears from Fig. 6.14 that P2-CP-CA sustained far larger displacement in this direction of loading. This was expected as a result of the lower axial load in P2-CP-CA. It is difficult to compare the pull response of P2-CP and P2-CP-CA because P2-CP test results in that direction were significantly influenced by the testing regime which consisted in pull direction testing upon completion of push direction testing. It is of considerable
interest that P2-HB performed similar to P2-CP in the push direction and possibly better than P2-CP in the pull direction. The suggestion is that using a single high strength block in each wall corner performs as well as the confining plate solution. Embedding high strength blocks in the lower corners appears to be a less laborious solution compared with embedment of confining plates.

Comparing the responses for P2-CP-CA and P2-CP-CA-ED in Fig. 6.15, it is clear that the latter developed the highest strength. This was due to the energy dissipators built into P2-CP-CA-ED that had a nominal yield strength of $2 \times 100 \text{kN} = 200 \text{kN}$. Adding this force to the applied prestressing force of approximately 600 kN suggests that the P2-CP-CA-ED wall strength would be approximately $(600 \text{kN} + 200 \text{kN})/600 \text{kN} = 1.33$ times higher than that of P2-CP-CA. This is confirmed by Fig. 6.15 and Table 6.3. The displacement capacity of P2-CP-CA-ED was lower than that of P2-CP-CA in the push direction (CP100 masonry), a direct effect of the higher stress imposed on the compression zone by the addition of energy dissipation bars. It appears that the displacement capacity of P2-CP-CA-ED in the pull direction inexplicably was superior to that of P2-CP-CA. Comparison of the performance of CP200 vs. CP100 for these two tests favours CP100 in terms of displacement capacity.
6.5.2 Tendon yielding

Tendon yielding was expected for all walls without axial load control. Significant prestress loss occurred for these walls, as is illustrated in Fig. 6.8. At the peaks of excursions P1-CP and P2-CP generally developed a total tendon force higher than the initial tendon force (Figs. 6.8(a) and (b)). P2-HB exhibited some tendon force increase at peak response. However, as wall toe degradation initiated, the total tendon force at displacement peaks started to decline to levels below the initial total tendon force. Despite tendon force loss due to yielding, the walls continued to return to their initial alignment (disregarding sliding), as suggested in Fig. 6.5.

No tendon yielding occurred for P2-CP-CA and P2-CP-CA-ED as a result of active control of the tendon force. Modest tendon force variation was recorded as shown in Figs. 6.8(c) and (d) as a result of friction in the hydraulic system.

6.5.3 Vertical masonry strain

Table 6.4 shows the vertical strain recorded in the plate confined walls at different stages of the tests. It is seen that the calculated masonry strains corresponding to the nominal flexural condition, $V_n$, ranged from 0.0019 to 0.0055 for CP200 and 0.0016 to 0.0027 for CP100. These results do not conform with the code defined nominal flexural strength condition set out by NZS 4230 for confined masonry, that specifies a useable extreme fibre masonry strain at nominal flexural strength of 0.008.

The masonry strain recorded at $V_{max}$ ranged from 0.0052 to 0.0190 for CP200 and 0.0037 to 0.023 for CP100. Thus a large variation in vertical strain at this limit state. It was felt at this limit state that the masonry fibres in the extreme ends of the walls still provided some axial strength, resisting the overturning moment.

Table 6.4 indicates that masonry extreme fibre strains ranged from 0.024 to 0.070 for CP200 and from 0.033 to 0.068 for CP100 at the ultimate displacement, $d_u$. These values are of course of theoretical character as the toe regions were damaged at this stage, so that those strains should not be regarded as the reliable ultimate strain capacity of concrete masonry. The results do nevertheless suggest that masonry strains far beyond those related to nominal flexural strength can be expected for rocking wall systems, while still providing significant axial strength. At this limit state the effective flexural neutral axis at the wall base had migrated towards the middle of the wall. Clear evidence of this is found in Figs. 6.9(b) through 6.13(b),
showing that the crossing of the strain profiles with the zero strain axis moved closer to the wall centre as the walls were displaced to larger and larger displacements.

6.5.4 Hysteretic energy dissipation

Comparison of Figs. 6.5(c) and (d) illustrates the effect of the energy dissipation bars in P2-CP-CA-ED. The area of all displacement cycles for P2-CP-CA-ED were increased considerably by yielding of the energy dissipation bars, as significant yielding commenced going to displacement level 4 (17 mm). The energy dissipated in each excursion (half cycle) due to hysteresis can be quantified by integration of the area enclosed by the loading and unloading curves. Results of such integration applied on the P2-CP-CA and P2-CP-CA-ED are illustrated in Fig. 6.16. The figure shows the cumulated energy dissipation as a function of the wall lateral displacement history. Fig. 6.16(a) illustrates that the energy applied to displace P2-CP-CA from zero displacement (original alignment) to the peak of an excursion, nearly was recovered upon subsequent unloading. On the contrary, the P2-CA-ED response shown in Fig. 6.16(b) suggests that only about 30-50% of the energy exerted by the loading system displacing the wall from zero displacement to a displacement peak was recovered upon subsequent unloading. As a numerical example, the total energy dissipated in the last two cycles shown in Figs. 6.16(a) and (b) has been calculated. The circular and square markers illustrate the beginning and end of the two displacement cycles to ±30-35 mm, respectively. Calculation shows that 4.8 kJ was dissipated by P2-CP-CA while 15.5 kJ was dissipated by P2-CP-CA-ED. Clearly, the hysteretic energy dissipation was increased substantially by the energy dissipation bars. Little sliding was measured for the walls depicted in Fig. 6.16 in the displacement range shown, thus this was not a source of 'parasitic' energy dissipation.
6.5.5 Sliding propensity

Wall sliding was measured for all walls, except P2-CP-CA-ED. This phenomenon was sought eliminated by the addition of sliding blocking plates placed adjacent to each end of the walls and by embedment of dowel bars. Testing results of P1-CP and P2-CP with sliding blocking plates mounted on the foundation showed that solution inadequate, because, once distress of the compression zone had initiated, little lateral force could be resisted by the compression zone as the wall slid towards the sliding blocking plates. Sliding for P2-HB did not initiate before integrity of the high strength blocks was lost. This occurred after approximately 40 mm of displacement. Again, the sliding blocking devices did not offer much restraint.

Walls P2-CP-CA and P2-CP-CA-ED had dowel bars embedded in both compression zones of the wall as shown in Fig. 6.1. These bars offered sufficient sliding restraint for P2-CP-CA in the pull direction (CP200), however when the CP200 end had failed and monotonic testing in the push direction only was instated for the remainder of the test, sliding initiated gradually as the CP100 compression zone integrity gradually degraded. It appears that the dowel bars worked satisfactorily in the push direction until displacements of about 45 mm (1.6% drift) was reached.

It is concluded that adequate shear friction between wall and foundation requires intentional roughening of the wall to foundation interface (refer to Chapter 8 for an in-depth discussion). Unbonded dowel bars, possibly in conjunction with longer continuous bars (in this case energy dissipation bars), appeared a viable option for adding further sliding resistance to the shear friction provided by the masonry/concrete foundation interface.

6.5.6 Initial stiffness

Estimation of initial stiffness is of considerable interest for serviceability loading. Fig. 6.17 shows curves for all P2 walls from Series 2 testing and for two walls from Series 1 testing, P2C and P2E (refer section below and to Chapter 5), relating to the wall secant stiffness at the theoretical first cracking limit state (refer to Chapter 8). It is seen in the figure that the initial stiffness varies little between the walls. This is not surprising because the wall initial stiffness theoretically only depends on the wall dimensions and the masonry elastic properties, and these were nearly identical for all tests. The average initial stiffness for the walls shown in Fig. 6.17 was 238 kN/mm compared to a theoretical elastic stiffness of 354 kN/mm (based on Eqns.
6.5.7 Comparison with Series 1 wall tests

The primary goal with this testing series was to investigate several means of strengthening the compression toe regions in order to improve wall displacement capacity. In this section comparison is made with two test units from Series 1 testing: FG:L3.0-W15-P2C and FG:L3.0-W15-P2C. Both walls had two prestressing bars embedded, the P2C bars concentrically at locations identical to all Series 2 wall tests (±400 mm) and the P2E bars eccentrically at ±1200 mm, and both walls featured unconfined ordinary masonry units throughout (U200). P2C results should be used for direct comparison with P2-CP and P2-HB because of similar dimension and prestressing layout. Fig. 6.18 shows the force-displacement envelopes for P2-CP, P2-HB, P2C and P2E.

Examining the wall responses in the push direction (positive displacement), the following observations are made: all walls developed similar strength; strength degradation for P2C and P2E initiated at approximately 0.4% drift while strength degradation for the other walls initi-

Fig. 6.17—Initial wall stiffness

8.2 and 8.4 using \( E_m = 15 \) GPa, \( v = 0.2 \) and \( P+N = 641 \) kN). The measured average wall initial stiffness was thus 33% lower than the theoretical one.
Fig. 6.18—F-D envelopes, P2-CP, P2-HB, P2C and P2E

ated at about 0.9% drift; the ultimate displacement capacity (at strength degradation to $0.8V_{\max}$ shown as dashed lines) for P2-CP, P2-HB and P2E were similar at approximately 1.2%, P2C had a displacement capacity of about 0.8%. It is therefore concluded that CP100 and HB solutions performed significantly better than U200 for both P2C and P2E in terms of initiation of strength degradation, and CP100 and HB performed better in terms of ultimate displacement capacity in comparison with U200 for P2C.

Examining the wall responses in the pull direction (negative displacement), similar observations are made: all walls developed similar strength (except for P2-CP that was influenced significantly by base sliding rendering the results difficult to compare with those from the other wall tests); rapid strength degradation for P2E initiated at approximately 0.3% drift while strength degradation of P2-HB and P2C was rather gradual; the ultimate displacement capacities (at strength degradation to $0.8V_{\max}$ shown as dashed lines) for P2-HB, P2C and P2E were approximately 1.4%, 1.1%, and 0.5%, respectively. It is therefore concluded that the HB solution performed significantly better than U200 for both P2C and P2E in terms of initiation of strength degradation and ultimate displacement capacity. It cannot be directly concluded from Fig. 6.18 that CP200 performed better than U200, however, the large displacement capacity in
the pull direction for P2-CP suggest that also CP200 masonry outperforms U200 masonry with a significant margin.

6.6 CONCLUSIONS

It is concluded that strengthening of the flexural compression zones of fully grouted unbonded post-tensioned concrete masonry walls successfully improved the wall displacement capacity and delayed the onset of strength degradation in comparison with a PCM wall of similar dimensions made with unconfined masonry. The maximum wall strength remained insensitive to strengthening of the compression zone.

The PCM walls reported on in this chapter behaved similarly to the behaviour observed for testing Series 1, notably they exhibited a nearly non-linear elastic behaviour dominated by rocking response. The exception was P2-CP-CA-ED that exhibited rocking response with non-linear elasto-plastic behaviour due to the presence of energy dissipation devices.

Large drift capacities were measured, 0.7% to 2% for directions with CP200 corners engaged, 1.1% to 2.6% with CP100 corners engaged and 1.1% to 1.4% with high strength block corners engaged. This clearly attests to consistent improvement of wall displacement capacity using confining plates or high strength corner units. The highest performing solution was CP100.

Only localised damage occurred, as shown in Fig. 6.6, making earthquake damage easy to repair. It is seen that nearly all damage occurred to the lowest masonry course for CP200 and CP100. This is supported by the vertical strain plots shown in Figs. 6.9 to 6.12, that suggest that little inelastic masonry response was measured above the lowest 200 mm of the walls. The high strength masonry units appear to have shifted masonry crushing to the second masonry course (U200) above the foundation because of the very high strength of the high strength units.

Use of confining plates is recommended for PCM wall construction based on the general observations of increase of wall displacement capacity and reduction of wall damage when compared with the performance of unconfined PCM walls. The use of high strength blocks in the lower wall corners appears to improve the wall performance. Still brittle masonry failure occurs in the masonry blocks just above the HB blocks which does not suggest increased wall toughness. The use of HB corners is not recommended for ductile design if HB blocks only are used to replace single masonry unit in the wall corners.
Relatively little energy dissipation was observed during cycling of all walls, except for P2-CP-CA-ED. This may impact the use of such walls for ductile seismic design in the sense that additional damping may be required to control the structural drift demand in an earthquake event. Therefore one wall had ‘dog-bone’ energy dissipation devices embedded. This simple device proved successful and worked as expected. A threefold increase of hysteretic damping was achieved at large displacement levels. The inherent simplicity and insensitivity to construction tolerance of the ‘dog-bone’ type device makes it a desirable option for ‘real life’ construction. Buckling restraint achieved by casing of the reduced diameter section of the ‘dog-bone’ energy dissipation bars with heavy pipe allows for placing of the energy dissipation bars inside the wall without risk of out-of-plane bursting of the masonry because of bar buckling.

Tendon force loss due to yielding should be expected at relatively low wall drift ratios, for squat walls. Even after tendon yielding, reliable and self-centring wall behaviour is expected. The hydraulic system used for control of the prestressing force for P2-CP-CA and P2-CP-CA-ED performed successfully.

Measurements at nominal flexural strength suggest that an extreme masonry fibre strain in the order of 0.0035 was found for CP200 and 0.0020 was found for CP100. Both values are substantially lower than 0.008 as stipulated by NZS 4230:1990 for confined masonry. At ultimate displacement, strains as high as 0.046-0.050 were measured for both CP200 and CP100. At this stage the extreme masonry fibres no longer carried axial load as face shell spalling had occurred.

It was concluded that adequate shear friction between wall and foundation requires intentional roughening of the wall to foundation interface. Dowel bars are effective for adding further sliding resistance.

Shear reinforcing should be embedded in PCM walls to reassure integrity of the masonry panel, though not necessarily needed for strength.
6.7 REFERENCES


Chapter 7

STRUCTURAL TESTING - SERIES 3

7. 3-STOREY PCM WALLS

Two 3-storey high unbonded post-tensioned concrete masonry (PCM) cantilever walls were tested in the Civil Test Hall at the University of Auckland. The 67% scale wall units were designed to model a typical cantilever wall from a 4-5 storey high office or apartment building. A detailed account of the wall construction, test setup, testing procedure and test results is provided in this chapter.

The principal intent with these wall tests was to validate the use of PCM in a realistic structural configuration. The test units, incorporating RC slabs at the intermediate floor levels, were subjected to a realistic moment gradient. Furthermore, the tests explored means of masonry confinement or strengthening that are expected to allow for reliable drift capacities beyond 1%.

7.1 INTRODUCTION

Previous testing by Laursen and Ingham, reported in Chapters 5 and 6, explored PCM walls made with unconfined and confined masonry. It was found that unconfined (ordinary) walls typically provided displacement (drift) capacities of up to 1%, however the limited strain capacity of unconfined masonry resulted in masonry crushing and subsequent strength degradation, and prohibited development of further reliable drift capacity. Using confined masonry, it was found that further displacement capacity could readily be developed, typically of the order of 1% to 1.5%, depending on the confinement method. As mentioned before, drift demands of such magnitude are likely to be encountered in structural systems utilising unbonded post-tensioned walls when subjected to seismic loading.

The objectives of this testing series were to:

Evaluate
1. Strength and displacement capacity of the walls
2. Effectiveness of using confinement plates in the bed joints to enhance the concrete masonry strain capacity

3. Energy dissipation during cyclic testing

**Determine**

1. Ultimate useable masonry strain
2. Vertical extent of the plastic deformation zone in the wall corners

**Compare**

1. Chapter 7 test results with testing of RCM walls of similar proportions

### 7.2 CONSTRUCTION DETAILS

#### 7.2.1 Wall specifications

The walls were designed to sustain large in-plane displacement arising from severe ground shaking. This was ensured by keeping the three prestressing strands (Fig. 7.1) unbonded over their entire length for maximum elastic elongation capacity and by enhancing the wall strength and strain capacity in the critical lower corners.

Wall dimensions are specified in Table 7.1. The wall designations do not follow the convention presented in the previous two chapters; the two walls are simply termed S3-1 and S3-2. Fig. 7.1 shows the wall dimensions and prestressing layout. The effective wall height, $h_e$, of 5,250 mm was determined by the maximum clear height in the Civil Test Hall. A storey height of 1,950 mm was chosen for the two lower levels, corresponding to approximately 67% scale model of a typical 3,000 mm storey height. Assuming that the effective wall height (equivalent height of the total seismic mass) amounts to about 2/3 of the total height, the test units correspond to 67% scale models of a 4 storey building of 12 m height. The wall length of 2,400 mm was cho-

<table>
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<tr>
<th>Wall</th>
<th>height $h_e$ (mm)</th>
<th>length $l_w$ (mm)</th>
<th>thickness $b_w$ (mm)</th>
<th>wall self weight $N_w$ (kN)</th>
<th>external weight $N_e$ (kN)</th>
<th>building weight $N$ (kN)</th>
<th>masonry strength $f'_m$ (MPa)</th>
<th>number of strands</th>
<th>initial prestress $P$ (kN)</th>
<th>$f_{ps}$ (kN)</th>
<th>axial load ratio $f_m/f'_m$</th>
</tr>
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<tbody>
<tr>
<td>S3-1</td>
<td>5,250</td>
<td>2,400</td>
<td>140</td>
<td>64</td>
<td>356</td>
<td>420</td>
<td>17.9</td>
<td>3</td>
<td>398</td>
<td>948</td>
<td>0.136</td>
</tr>
<tr>
<td>S3-2</td>
<td>5,250</td>
<td>2,400</td>
<td>140</td>
<td>64</td>
<td>142</td>
<td>206</td>
<td>13.6</td>
<td>3</td>
<td>427</td>
<td>1017</td>
<td>0.139</td>
</tr>
</tbody>
</table>

*initial, 'unconfined strength, half height units, $T_n = (P+N)/(l_w b_w)$
Fig. 7.1—Wall geometry for Series 3 walls

Sen to reflect a realistic wall aspect ratio (height/length) of approximately 3.4. Using 15 series concrete masonry blocks with nominal thickness of 140 mm, the wall test units represented 67% scale models of 209 mm thick walls. This appears to be a realistic wall thickness that could be achieved with readily available 20 or 25 series concrete masonry blocks. The wall dimensions and layout could also represent full scale walls of 5.25 m height using 140 mm masonry.

Ordinary 15 series concrete masonry blocks, Trade Mortar™ and 17.5 MPa grout were used for construction. Confining plates were embedded in all bed joints in the lower 1,000 mm of
Fig. 7.2—Dimensions and location of confining plates

the walls and extended approximately 600-800 mm from the wall ends. Fig. 7.2 shows the dimensions and locations of the embedded confining plates, referred to as CP100 masonry when embedded in bed joints between half height masonry units. The plates were welded from 25 mm by 3 mm mild steel flat and proportioned such that grout continuity was not impeded. Research presented in Chapter 3 and Chapter 6 has shown that such a system confines the grouted concrete masonry, increasing the axial strength and strain capacity.

Both walls were prestressed with three 15.2 mm high strength strands placed at ±400 mm and 0 mm from the wall centre line. The prestressing tendons remained unbonded over the entire wall height and had a typical unbonded length of 6,400 mm, including unbonded lengths in the loading beam (300 mm), foundation (300 mm), strong floor (400 mm) and load cells underneath the strong floor (300 mm).

The initial prestressing force for both walls was determined based on a required wall drift of 2% before tendon yield assuming that the wall simply rocks around the lower corners. 2% drift is a typical code defined maximum and corresponds to a wall displacement of 105 mm at the equivalent height of $h_e = 5.25$ m. This resulted in a target initial prestressing steel force of 400 kN or approximately $0.55f_{pu}$. The achieved wall initial prestressing force and axial force ratios (due to prestressing and building weight) are given in Table 7.1.
Horizontal shear reinforcing was provided at 400 mm vertical spacing. Each level consisted of two HD10 deformed bars (10 mm deformed bars, 430 MPa yield strength) placed one on each side of the prestressing ducts.

For S3-1, self-weight and tributary loads from supported floors was stipulated to be 1.0 MPa masonry stress or 336 kN at the wall base, corresponding to a total building weight of 400 kN. As the wall self-weight of the tested assembly (masonry and RC concrete) was approximately 64 kN, an additional externally applied axial force of 336 kN was required (refer to external axial load system discussed in section 7.3.1). Lighter axial loading was assumed for wall S3-2 such that the total building weight amounted to 200 kN (0.5 MPa masonry stress), requiring 136 kN of additional axial force to be applied. This corresponded to the wall strength (restoring moment) due to prestressing (P) and building weight (N) being provided on a 50%:50% basis for wall S3-1 and on a 67%:33% basis for S3-2.

7.2.2 Wall construction

The walls were constructed in running bond by experienced blocklayers under supervision, using standard grey precast concrete masonry units and DRICON™ trade mortar. Open-ended concrete masonry units were used to avoid having to thread the masonry units onto the ovalised prestressing ducting, see Fig. 7.3(a); note the HD10 (deformed 10 mm bar, $f_y = 430$ MPa) shear reinforcing bars placed on each side of the oval duct. Confining plates, as shown in Figs. 7.2 and 7.3(b), were embedded in the zone of half height masonry units with mortar on both sides for best integrity. Grouting with 17.5 MPa target strength ready-mixed regular block fill, containing SIKA CAVEX™ expansion agent, was performed the subsequent day. Vibration of the grout was carried out for all grout pours. Wash-out ports were provided on the north face in the lowest halfheight block in each corner.

Construction of wall S3-1 proceeded in four phases:

1. The first storey was constructed on the wall foundation situated in the testing rig. After grouting and curing of the first storey, a RC floor slab was placed on top of this storey on a mortar pack.

2. The second storey was next built in the test setup on top of the first storey RC slab. Meanwhile, the third storey was constructed on the second storey RC floor slab temporarily placed on the laboratory floor.
3. After grouting and curing of the second and third storeys, the third storey assembly was placed on top of the first and second storeys in the test setup. Vertical alignment of the three stories was simply ensured using a plumb bulb.

4. Placing of the reinforced concrete loading beam on a mortar pack, and installation and stressing of the prestressing strands.

Construction of wall S3-2 proceeded differently. Only the bottom part of the first storey wall of S3-1 had been damaged during testing, therefore, only the bottom storey wall was replaced for S3-2 in the following steps:

1. 2\textsuperscript{nd} and 3\textsuperscript{rd} storey walls, including RC slabs and loading beam, were lifted off the first storey wall using chain blocks attached to the testing rig.
2. The damaged first storey wall was removed.
3. The first storey was reconstructed on the wall foundation and grouted.
4. Upon curing, the second and third storey assembly was repositioned on top of the first storey wall on a mortar pack.

The reinforced concrete foundation, floor slabs and loading beam were manufactured of 30 MPa target strength concrete, prior to masonry construction. Fig. 7.1 shows the concrete dimensions and Fig. 7.4 shows the reinforcing layout.
7.2.3 Material properties

Average masonry material properties were determined by material testing, typically in samples of three. The masonry crushing strength $f_m'$, masonry elastic modulus $E_m$ (masonry: 600 mm $\times$ 400 mm $\times$ 140 mm prism), grout crushing strength $f_g'$, mortar crushing strength $f_m'$ (grout and mortar: 200 mm high $\times$ 100 mm diam. cylinder), and average age at day of testing are given in Table 7.2. Average strengths of concrete were 38 MPa for foundation and 1st floor slab and 49 MPa for 2nd floor slab and loading beam.

Seven-wire prestressing strand of the specification ‘AS 1311-1987, Super, Low Relaxation’ was used for both wall tests. Nominal properties of the 15.2 mm diam. prestressing strand were
tensile strength (rupture strength) of 250 kN ($f_{pu} = 1785$ MPa), yield strength of 213 kN ($f_{py} = 1520$ MPa), minimum ultimate elongation capacity of 3.5%, elastic modulus $E_{ps} = 190$ GPa, maximum relaxation after 1000 hours of 2.5% and tendon area of 140 mm$^2$. Proof testing of a strand sample by the manufacturer to AS1311-1987 [7-1] confirmed the above properties with the following results: $f_{py} = 252-257$ kN, $f_{pu} = 271-274$ kN, elongation of 7.0-7.7% and $E_{ps} = 191-192$ GPa.

### 7.3 TESTING DETAILS

#### 7.3.1 Test setup

Wall testing reported in this chapter was conducted using the setup shown in Fig 7.5, consisting in principle of the test unit, a foundation beam, a loading beam, a horizontal hydraulic actuator and a reaction wall. The foundation beam was clamped to the floor with stress bar and the loading beam clamped to the top of the wall by the three vertical prestressing strands. The hydraulic actuator applied lateral load at a nominal height of 5,250 mm above the foundation beam, and was prestressed to the loading beam by means of two 23 mm VSL CT stress bars that spanned the full length of the loading beam.

Addition axial load, modelling gravity and live loads from suspended floors, was applied by means of the external axial load system shown schematically in Fig. 7.6. The system consisted of two vertical 23 mm VSL CT stress bars spanning between the foundation beam and the steel cross-beam. Hydraulic hollow jacks applied axial force in the bars. The jack shown on the left hand side (north face) was connected to a hydraulic pump providing constant pressure thus constant force. The cross beam was free to pivot about a horizontal axis parallel to the plane of the wall, thus equalising the force in the two external bars. The jack on the right hand side (south face) was passive and was used solely to adjust the pivot angle of the cross-beam. As the wall was displaced laterally and rocking action extended the distance between the foundation

### TABLE 7.2—Material properties

<table>
<thead>
<tr>
<th>Wall</th>
<th>$f_m$</th>
<th>$f_g$</th>
<th>$f_g$</th>
<th>$f_j$</th>
<th>Age at DOT</th>
<th>$f_m$</th>
<th>$f_g$</th>
<th>Age at DOT</th>
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<td>U200</td>
<td>CP100</td>
<td>Cavex</td>
<td>No Cavex</td>
<td>mortar</td>
<td>DOT</td>
<td>DOT</td>
<td>DOT</td>
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<td>17.9</td>
<td>18.1</td>
<td></td>
<td>12.7</td>
<td>-</td>
<td>11.3</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
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<td>MPa</td>
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<td>MPa</td>
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<td>days</td>
</tr>
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<td>MPa</td>
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<td>days</td>
</tr>
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</table>
Fig. 7.5—Testing setup
and the loading beam, the left hand side jack length automatically adjusted accordingly so that the external axial load was kept constant.

Fig. 7.7 shows a photograph of the south face of wall S3-1 in the testing setup, ready for testing. Lateral restraint was provided to the wall at RC floor slab and loading beam levels by means of horizontal steel sections on both sides of the wall, spanning between the black column in the foreground and the strong wall extension in the background. Steel rollers were attached to the RC slabs, two on each side, and bore against the horizontal beams. Lateral restraint to the loading beam was provided by low friction Teflon to stainless steel interfaces, two on each side.

7.3.2 Instrumentation
Fig. 7.8 schematically shows the wall instrumentation. Lateral displacement was measured at floor and loading beam levels as shown in Fig. 7.8(a). Instrumentation was tripled at each level, consisting of portal gauges (PG for small displacements), turn potentiometers (TP for large displacements) and digital display extensiometers (DG for verification of PG and TP). These
instruments were mounted on an independent instrument column which is seen between wall and strong wall extension in Fig. 7.7. Horizontal load was measured by the load cell LCH positioned in series with the lateral hydraulic actuator, see Fig. 7.8(b). Force in the prestressing strand was measured by load cells LCP1, LCP2 and LCP3 near the anchorage point in the test hall basement. Applied external axial load was measured by load cells LCAN and LCAS.

Fig. 7.8(c) shows flexural instrumentation FN (north face) and FS (south face) of the portal gauge type used for measuring vertical strain throughout the wall. Portal gauges FN01-05, FN08-27, FN30, FN38, FS01-05, FS08-27, FS30 and FS31 had a nominal gauge length of 200 mm; all other FN and FS portal gauges had a nominal gauge length of 400 mm. All FN and FS portal gauges spanned between bed joints. Sliding displacement was monitored by the instrumentation SL shown in Fig. 7.8(d).

7.3.3 Testing procedure

The walls were subjected to cyclic in-plane loading applied to the loading beam located approximately 5,250 mm above the wall base (see Fig. 7.5). The testing procedure adopted for
these tests was based on that adopted by Rahman and Restrepo [7-2] and Holden [7-3], and consisted of two loading regimes:

(i) Force controlled load cycles for serviceability loads with forces limited by the nominal flexural strength. These force cycles consisted of one cycle (one push and one pull excursion) to the wall cracking strength, to the wall maximum serviceability strength (maximum allowed serviceability limit state concrete masonry stress), and to 75% and 100% of nominal flexural strength.

(ii) Displacement-controlled cyclic loading, where the wall was cycled twice to the drift target, followed by one cycle to the previous drift target. In this chapter drift is defined as the ratio of the top lateral displacement d to the wall height $h_c$: $\gamma = d/h_c$. Displacement to drift levels rather than displacement ductility was chosen in the spirit of performance based design, where the drift ratio appears to be a suitable design parameter.
7.3.4 Flexural strength prediction

For a symmetric distribution of post-tensioning and gravity actions, first cracking strength, $V_{cr}$, maximum serviceability strength, $V_s$, and the nominal flexural strength, $V_f$, are defined by Eqns. 8.2, 8.6 and 8.17, respectively. The base shears, $V_{cr}$, $V_s$ and $V_f$, correspond to the calculated flexural moment at the wall base. For nominal flexural strength, the tendon force increase $\Delta P$ due to wall deformation was calculated using Eqns. 8.25 and 8.26. The equivalent rectangu-
lar compression zone length, $a$, defined by Eqn. 8.18 was calculated using $\alpha = 0.96$ (confined grouted concrete masonry). Table 7.3 reflects the calculated strengths.

7.3.5 Predicted masonry shear strength

The predicted concrete masonry shear strength, $V_m$ (force), was calculated according to Eqn. 5.2. Shear strength due to shear reinforcing, $V_{sr}$, was calculated using Eqn. 6.1. The predicted
wall shear strengths \( V_s = V_m + V_u \) are listed in Table 7.3. The initial prestress force was used in this calculation.

### 7.4 TEST RESULTS

This section describes the observed behaviours of S3-1 and S3-2 and presents the measured masonry compression toe strains at various wall states. General wall behaviour is summarised in Table 7.4, where \( V_{\text{max}} \) is the maximum lateral force recorded and \( d_{\text{vmax}} \) is the corresponding displacement. The ultimate displacement capacity, \( d_u \), is defined as the point at which the lateral wall strength had degraded below 80% of \( V_{\text{max}} \). The ultimate drift capacity is defined as \( \gamma_u = d_u / h_e \) with \( h_e = 5.25 \) m.

Rocking response was recorded for both wall tests, with a single large crack opening up along the wall-foundation construction joint. No distributed flexural cracks were observed. Additionally, singular horizontal cracks developed at the interface between the bottom storey and the level 2 floor slab for both walls as a result of flexural action. These cracks developed at loading level \( V_f \) for S3-1 and 0.25% drift for S3-2, extending 400-600 mm from the wall ends, and closed fully upon unloading. At force levels near nominal flexural strength, \( V_f = M_{\text{u}} / h_w \), wall

### TABLE 7.3—Strength predictions

<table>
<thead>
<tr>
<th>Wall</th>
<th>N</th>
<th>P</th>
<th>( V_{\text{cr}} )</th>
<th>( V_e )</th>
<th>( V_f )</th>
<th>( \Delta P )</th>
<th>( V_s )</th>
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<td>62</td>
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<td>165</td>
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<td>673</td>
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<td>S3-2</td>
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<td>427</td>
<td>48</td>
<td>98</td>
<td>129</td>
<td>21</td>
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### TABLE 7.4—Predictions and results

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<th>direction</th>
<th>( V_{\text{max}} )</th>
<th>( d_{\text{vmax}} )</th>
<th>( y_{\text{vmax}} )</th>
<th>( d_u )</th>
<th>( \gamma_u )</th>
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<tr>
<td>S3-1</td>
<td>165</td>
<td>Push</td>
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<td>79.2</td>
<td>1.51</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>Pull</td>
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<td>0.70</td>
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<td>1.28</td>
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<td>Push</td>
<td>174</td>
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<td>1.24</td>
<td>79.8</td>
<td>1.52</td>
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</tr>
<tr>
<td></td>
<td></td>
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<td>1.04</td>
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- 158 -
softening occurred due to extensive opening of the base crack and the initiation of rocking. Tendon yielding was not experienced.

The failure mode was characterised by localised masonry crushing in the compression toe regions, resulting in gradual strength degradation. Visual inspection showed that using CP100, inelastic vertical masonry strain had penetrated into the third course above the foundation (0-300 mm). The cycles to drifts of 0.25% to 0.5% revealed little evidence of distress of the masonry compression zones. First visual indication of fine vertical splitting cracks in the face shells of the first masonry course was observed at 0.5% drift. During cycles to drifts of 0.75% and 1.0% some spalling of mortar in the lower bed joints was observed and vertical cracks occurred in the masonry face shell at each wall end in the first course (100 mm). No strength degradation was observed at this stage.

Negligible sliding displacements were measured and there was no indication of friction created by the lateral restraint devices due to out-of-plane wall displacement.

7.4.1 Force-displacement response

Fig. 7.9, depicting the force-displacement response for S3-1 and S3-2, demonstrates the nearly non-linear elastic behaviour exhibited by the tested unbonded post-tensioned walls prior to toe crushing. It is seen that the walls returned to their original alignment, even after large displacement excursions. This type of response may be categorised as ‘origin oriented’. The individual curves for each excursion appear pinched, implying little hysteretic energy dissipation. It is noted that the term ‘elastic’ in a rigorous sense indicates reversibility and no accumulation of damage. In reality the walls did accumulate damage from the onset of masonry crushing, causing a reduction in wall stiffness and some hysteretic energy dissipation. Thus the behaviour was termed ‘nearly’ non-linear elastic.

Significant strength degradation for both walls occurred in the first excursion cycle to ±1.5% drift. With the intent of displacing S3-2 to -1.5% drift, the wall was accidentally displaced much further to -2.1% drift, resulting in extensive crushing of the compression zone and subsequent severe strength loss for the pull direction.

Notably, neither wall exhibited symmetrical response in the push and pull directions, with the maximum push strength being 8-10% higher than the maximum pull strength. This discrepancy was recorded despite symmetric strengthening of the wall corners as indicated in Fig. 7.2.
Fig. 7.9—Force-displacement histories
It is seen in Fig. 7.9 that both S3-1 and S3-2 developed significantly higher strength than predicted (Vf), specifically 21% and 28% more than Vf for S3-1 and S3-2, respectively.

Fig. 7.10 shows the loading histories for the test units. The two walls were taken through similar cyclic displacement patterns with a few exceptions, most notably that S3-2 additionally was taken through two cycles to ±1.25% drift.
7.4.2 Damage pattern and failure mode

Gradual strength degradation was observed for both walls and was attributed to spalling of face shells and crushing of the grout core. Rupture of the confining plates was not observed but some distortion of the shape occurred in the bed joints in the crushed corners. Large displacement capacity was observed with reliable wall resistance for drifts, $\gamma_u$, ranging from 1.3% to 1.5% for S3-1 and from 1.5% to 1.8% for S3-2, refer to Table 7.4. Damage patterns after failure are depicted by the photographs in Fig. 7.11, showing both sides of the lower storey of both walls. The photographs indicate that the extent of damage was confined to the lowest three masonry courses (300 mm) in the toe regions, with the majority of damage concentrated in the lowest two courses (200 mm). The damage to the masonry units immediately above the third course (300 mm and further above base) generally was limited to vertical splitting cracking near the wall end. No cracking, whatsoever, was observed in the upper two stories.

7.4.3 Sliding

Fig. 7.12 presents the recorded sliding displacement for the two wall tests. It appears from Figs. 7.12(a) and (b) that sliding increased gradually with lateral displacement. The maximum sliding displacement was of the order of 1.5 mm to 2.5 mm, or less than 3% of the total lateral displacement measured at any excursion peak. It is therefore concluded that base sliding displacements were insignificant in comparison with the total lateral displacement and that the shear friction between the intentionally roughened foundation and the masonry (cleaned for mortar droppings before grouting) was adequate to resist sliding.

7.4.4 Prestressing force

The prestressing tendon force histories (sum of force of all tendons) for both walls are plotted against the lateral displacement in Figs. 7.13(a) and (b). For reference, it is noted that the nominal yield strength for one strand is about 213 kN.

These figures reveal that the prestressing force at all excursion peaks exceeded the initial prestressing force and that little tendon force loss occurred until the detrimental excursions to $\pm1.5\%$ drift. Recorded data showed that all prestressing strands remained elastic throughout testing. After failure prestress losses of 8% and 10% were recorded for S3-1 and S3-2, respectively.
Fig. 7.11—(a) and (b), S3-1 Damage accumulation at failure
Fig. 7.11—(c) and (d), S3-2 Damage accumulation at failure
Fig. 7.12—Sliding displacement
Fig. 7.13—Prestressing force histories
7.4.5 External axial force

Figs. 7.14(a) and (b) show the variation of the externally applied axial force for S3-1 and S3-2. The target forces and typical excursion peak forces are shown in the figures and indicate that approximately 30 kN more than the target force typically was applied for both walls. This corresponds to a total axial load increase (N+P) of approximately 3.8% and 5.0% for S3-1 and S3-2, respectively. The external axial force variation was attributed to friction in the hydraulic jack.

7.4.6 Vertical masonry strain

Vertical strain along the bottom of the walls recorded during testing is shown in Figs. 7.15 and 7.16. Each figure (plots a through h) shows strain plots at displacement peaks for the first cycle to a given lateral drift for instrument levels 1 through 4 above the base, i.e. average strain for 0-200 mm, 200-400 mm, 400-600 mm and 600-800 mm above the foundation. As flexural instrumentation was mounted on both sides of the wall, the average strain at each location was used. The sign convention in these figures defines compression strain as negative. The vertical thick lines in the figures indicate the wall extremities, i.e. the extreme masonry fibres at ±1.2 m from the wall centre line. Plots (a), (c), (e) and (g) relate to low displacement response, excursions to $V_{cn}$, $V_e$, $V_f$ and approximately 0.3% drift, and plots (b), (d), (f) and (h) relate to response beyond 0.3% drift. Strain measurements for these walls were recorded between bed joints of two courses of CP100.

No plots of vertical strain above level 4 have been included in this report because the recorded vertical strain indicated purely elastic response (vertical strain significantly lower than 0.001). This attests to the vertical extent of the compression zone undergoing plastic deformation remained rather short. Furthermore, the resolution of the utilised instruments of perhaps 0.0002 strain (using a gauge length of 200 mm) excluded meaningful interpretation of data recorded above level 4. It is remarked that the data plotted in Figs. 7.15 and 7.16 is associated with some degree of uncertainty, especially at high drift levels where deterioration of the masonry unit face shells may have affected instrument reading.

Figs. 7.15(a)+(b) and 7.16(a)+(b) reveal that vertical strain varied nearly linearly along the length of the walls base for all displacement levels.
Typical load at excursion peak 370 kN

Target 340 kN

(b) S3-2

Fig. 7.14—External axial load histories
Fig. 7.15—S3-1, Horizontal strain profiles, levels 0-200 mm and 200-400 mm
Fig. 7.15—(Cont.) S3-1, Horiz. strain profiles, levels 400-600 mm and 600-800 mm
Fig. 7.16—S3-2, Horizontal strain profiles, levels 0-200 mm and 200-400 mm
Fig. 7.16—(Cont.) S3-2, Horiz. strain profiles, levels 400-600 mm and 600-800 mm
The general trends for masonry strain for low level response (plots (a)) were extreme fibre strains of 0.0010 to 0.0020 for readings up to \( V_r \) and approximately 0.0020-0.0030 for 0.25%-0.3% drift. The extreme fibre strain for large displacement response ranged from approximately 0.02 to 0.04. Extreme strains at levels 2 to 3 exceeded 0.0015 strain indicating the masonry responded inelastically. Some inelastic response (strain higher then 0.0015) was also recorded for S3-2 level 4. Note that the above considerations of extreme strain at wall ends (±1.2 m) were based on extrapolation of recorded strain at lateral locations ±0.9m and ±1.1 m.

Table 7.5 presents a summary of vertical strain measurements and vertical dimension of the plastic region for the extreme masonry fibre, related to drift level peaks. The extreme masonry strain was found by extrapolation of the results presented in Figs. 7.15 and 7.16 to the wall extreme ends at ±1.2 m. Determination of the vertical extent \( h_p \) of inelastic masonry response was based on a reference strain of 0.0015, such that a masonry instrument level with average extreme fibre strain measurement exceeding the reference strain was deemed inelastic. The

**TABLE 7.5—Vertical strain**

<table>
<thead>
<tr>
<th>Wall</th>
<th>Push drift</th>
<th>strain</th>
<th>( h_p )</th>
<th>event</th>
<th>Push drift</th>
<th>strain</th>
<th>( h_p )</th>
<th>event</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-1</td>
<td>( V_f )</td>
<td>0.0010</td>
<td>0</td>
<td></td>
<td>( V_f )</td>
<td>0.0014</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.26%</td>
<td>0.0020</td>
<td>200</td>
<td></td>
<td>0.24%</td>
<td>0.0024</td>
<td>200</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.53%</td>
<td>0.0037</td>
<td>400</td>
<td></td>
<td>0.47%</td>
<td>0.0032</td>
<td>400</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.71%</td>
<td>0.0057</td>
<td>400</td>
<td>( V_{max} )</td>
<td>0.62%</td>
<td>0.0035</td>
<td>600</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.06%</td>
<td>0.0120</td>
<td>400</td>
<td></td>
<td>0.96%</td>
<td>0.0056</td>
<td>600</td>
<td>( V_{max} )</td>
</tr>
<tr>
<td></td>
<td>1.31%</td>
<td>0.0226</td>
<td>400</td>
<td>( \gamma_u )</td>
<td>1.23%</td>
<td>0.0126</td>
<td>400</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.52%</td>
<td>0.0362</td>
<td>400</td>
<td></td>
<td>1.51%</td>
<td>0.0193</td>
<td>400</td>
<td>( \gamma_u )</td>
</tr>
<tr>
<td>S3-2</td>
<td>( V_f )</td>
<td>0.0022</td>
<td>200</td>
<td></td>
<td>( V_f )</td>
<td>0.0012</td>
<td>0</td>
<td></td>
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<tr>
<td></td>
<td>0.31%</td>
<td>0.0033</td>
<td>200</td>
<td></td>
<td>0.30%</td>
<td>0.0024</td>
<td>200</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.51%</td>
<td>0.0036</td>
<td>400</td>
<td></td>
<td>0.44%</td>
<td>0.0025</td>
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</tr>
<tr>
<td></td>
<td>0.75%</td>
<td>0.0043</td>
<td>400</td>
<td></td>
<td>0.68%</td>
<td>0.0038</td>
<td>600</td>
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<tr>
<td></td>
<td>1.02%</td>
<td>0.0066</td>
<td>400</td>
<td>( V_{max} )</td>
<td>0.99%</td>
<td>0.0057</td>
<td>600</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.27%</td>
<td>0.0099</td>
<td>400</td>
<td>( V_{max} )</td>
<td>1.22%</td>
<td>0.0079</td>
<td>600</td>
<td></td>
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<td></td>
<td>2.12%</td>
<td>0.0437</td>
<td>400</td>
<td>( \gamma_u )</td>
<td>1.50%</td>
<td>0.0242</td>
<td>400</td>
<td>( \gamma_u )</td>
</tr>
</tbody>
</table>

% | mm | % | mm |
extreme strain presented in Table 7.5 reflects the average of the strains measured by the instruments within the vertical extent of h_p. The following was observed from Table 7.5: (1) vertical strain at nominal flexural strength V_f indicates elastic masonry response (in italic font) at this load level (with the exception of S3-2 pull), (2) strain at V_{max} ranged from 0.0056 to 0.0079, (3) strain at γ_0 ranged from 1.9% to 2.4%, (4) h_p at V_{max} measured 400 mm for pull excursions and 600 mm for push excursions and (5) h_p at γ_0 measured 400 mm for both walls.

7.4.7 Lateral wall displacement profile

Figs. 7.17 and 7.18 show the lateral displacements at the RC floor slab and loading beam levels recorded at peaks of first excursions to the specified drift levels.

Studying the low level response plots (a), it is seen that both walls exhibited distinct flexural action for loading between V_{cr} and V_f. These displacement profiles are 'curved' with distinct 'kinks' at the first RC floor slab level, suggesting that a significant amount of the lateral displacement arose from flexural action throughout the wall. The curvature became less distinct as the walls were loaded to higher force.

Looking at the plots of high drift level lateral displacement profiles in Figs. 7.17(b) and 7.18(b), it is clear that rocking action at the base became the dominant source of lateral displacement. The lateral displacement profiles became straight lines and the 'kink' previously observed at the first RC floor slab level disappeared.

7.5 DISCUSSION

7.5.1 Flexural response

Fig. 7.9 indicates that the developed flexural strength V_{max} (base shear corresponding to the maximum moment) exceeded the predicted flexural strength V_f in both wall test, specifically S3-1 developed 21% higher strength (average of push and pull) than V_f, while S3-2 developed 28% higher strength than V_f. These measured strengths were surprisingly high because calculating the wall strength (base shear corresponding to the restoring moment due to P+N) using the measured peak values of P and N and α = 0.96 could not account for the measured strength in either of the wall tests. The author is confident that the lateral load (LCH) was measured within 5% of accuracy and that little horizontal resistance was caused by the lateral restraining devices. The load cells measuring the additional axial force (LCAN and LCAS) recorded simi-
(a) $V_{cr} - V_f$ (digital extensometers)

(b) All drift levels (PG and TP)

Fig. 7.17—S3-1, Lateral displacement profile
(a) $V_{cr}$-$V_f$ (digital extensometers)

(b) All drift levels (PG and TP)

Fig. 7.18—S3-2, Lateral displacement profile
Fig. 7.19—Force-displacement envelopes

lar force throughout the tests, suggesting that their measurements were reliable. Apart from construction inaccuracies, e.g. location of prestressing strand and external axial force device, the last source of error could be attributed to measurement of prestressing force. While the load cell readings (LCP1, LCP2 and LCP3) were considered reliable, the actual prestressing force between the bottom of the walls and the loading beam may have been higher than measured because of friction generated between the strands and the holes in the foundation beam and strong floor due to misalignment. This source of error was presumably not large because friction would have caused hysteresis that would have shown on the prestressing force histories in Fig. 7.12 as increased area between loading and unloading curves.

Comparing the force-displacement envelopes (peak response of first excursion cycle for each displacement level) for the walls, shown in Fig. 7.19, it is seen that S3-1 developed 21% higher strength than S3-2 (average of loading directions). This was a direct result of the total axial force on S3-1 being approximately 29% higher than that of S3-2.

Comparing the response envelopes in Fig. 7.19 and the results presented in Table 7.4, it is clear that S3-2 developed the largest drift capacity; S3-1 sustained 1.40% drift (average of loading
directions) while S3-2 sustained 1.65% drift. This was expected because of the lighter axial force applied to S3-2, despite the fact that the S3-2 masonry crushing strength was significantly lower than that of S3-1.

7.5.2 Tendon behaviour

Tendon yielding was not expected and did not occur, as is illustrated in Fig. 7.13. At the peaks of excursions both walls developed total tendon force higher that the initial tendon force. It appears from the recorded tendon force data that S3-1 developed a maximum tendon force (single tendon) of 198 kN while S3-2 developed a maximum tendon force of 209 kN. The nominal tendon yield force of 213 kN, specified by the strand manufacturer, was nearly reached during testing of S3-2. It is concluded that determining the initial prestressing force based on the assumptions of rigid body rocking behaviour and tendon yielding at 2% drift functioned as intended for both walls, maximising the tendon force (economy) while protecting the tendon against inelastic stress.

7.5.3 Vertical masonry strain

Table 7.5 shows the vertical strain recorded in the walls at various stages of the tests. It is seen that the calculated masonry strains corresponding to the nominal flexural condition, $V_f$, ranged from 0.001 to 0.0022. These results do not conform with the code defined nominal flexural strength condition set out by NZS 4230 for confined masonry, that specify a useable extreme fibre masonry strain at nominal flexural strength of 0.008.

The masonry strain in the plastic deformation zone ($h_p$) at $V_{\text{max}}$ ranged from 0.0056 to 0.0079, a small variation in vertical strain. The lack of visible damage at this limit state impelled that the masonry fibres in the extreme ends of the walls still provided some axial strength resisting the overturning moment. Stains at this limit state compared favourably with the code defined useable strain of 0.008.

Table 7.5 indicates that masonry extreme fibre strains ranged from 0.019 to 0.024 at the ultimate drift, $\gamma_u$, with values again showing little scatter considering the circumstances of measurement. These values are of course of theoretical character as the toe regions were damaged at this stage, so that those strains should not be regarded as the reliable ultimate strain capacity of concrete masonry. The results do nevertheless suggest that masonry strains far beyond those related to nominal flexural strength can be expected for rocking wall systems, while still pro-
viding significant axial strength. At this limit state the effective flexural neutral axis at the wall base had migrated towards the middle of the wall. Clear evidence of this is found in Figs. 7.15(b) and 7.16(b), showing that the crossing of the strain profiles with the zero strain axis moved closer to the wall centre as the walls were displaced to larger and larger displacements.

The height of the plastic deformation zone was tracked throughout testing and is presented in Table 7.5. Inelastic deformation occurred over a height of approximately 400-600 mm at $V_{\text{max}}$, while measurements showed that this height had shortened at ultimate drift. Some unloading of the masonry above 400 mm occurred at $\gamma_0$ due to crushing of masonry below. Nevertheless inelastic deformation had occurred over 400-600 mm.

### 7.5.4 Hysteretic energy dissipation

Figs. 7.9(a) and (b) illustrate that little hysteretic energy dissipation occurred in the two wall tests, until reaching the 1.5% drift level. Nevertheless, the energy dissipated in each excursion (half cycle) due to hysteresis can be quantified by integration of the area enclosed by the loading and unloading curves. Results of such integration applied on to the force-displacement relationships for S3-1 and S3-2 are illustrated in Fig. 7.20, showing the cumulated energy dissipation as a function of the wall lateral displacement history. As discussed above, the externally applied axial load varied during testing and therefore caused ‘parasitic’ energy dissipation. Calculations showed that the parasitic component of the energy dissipation amounted to less than 12% and 11% of the total hysteretic energy dissipation for S3-1 and S3-2, respectively. Plots (a) and (b) in Fig. 7.20 show the ‘true’ hysteretic energy dissipated by wall flexural action, calculated by subtraction of parasitic energy dissipation from total calculated energy dissipation.

It is seen in Fig. 7.20 that similar amounts of energy were dissipated by the two walls and that 85-90% of the energy that was exerted by the horizontal actuator to displace a wall from zero displacement to a displacement peak was typically released upon unloading, thus 10-15% of energy was lost in a single excursion. This can be compared to the elasto-plastic hysteretic behaviour of reinforced concrete masonry walls that dissipate energy in both the excursion segment to a displacement peak and the excursion segment that returns the wall to its original alignment. Positive work is required in both the loading and unloading branches of the excursion. Clearly PCM dissipates far less energy than reinforced concrete masonry for walls of similar characteristics.
Fig. 7.20—Cumulated hysteretic energy dissipation
7.5.5 Sliding propensity

Significant wall sliding was not measured as documented by Fig. 7.12. This phenomenon was overcome by adequate shear friction between wall and foundation provided by intentional roughening of the wall to foundation interface and cleaning of the wall cavities in the compression zone for mortar droppings. Refer to Chapter 8 for an in-depth discussion.

7.5.6 Initial stiffness

Estimation of initial stiffness is of considerable interest for serviceability and ultimate loading in relation with vibration characteristics and earthquake loading. Fig. 7.21 shows curves for both walls relating to the wall secant stiffness at the theoretical first cracking limit state. It is seen in the figure that the initial stiffness varied between the walls, however the average initial stiffness of both wall were similar, 41.1 kN/mm for S3-1 and 41.0 kN/mm for S3-2. This is not surprising because the wall initial stiffness theoretically only depends on the wall dimensions and the masonry elastic properties, and these were nearly identical for both tests, with the exception of the lowest storey where the elastic modulus of the masonry for S3-2 was some-
what lower than that of S3-1. The average initial stiffness for the walls shown in Fig. 7.21 of 41.1 kN/mm can be compared with the theoretical elastic stiffness of 44.1 kN/mm (based on Eqns. 8.2 and 8.4 using $E_m = 14.4$ GPa and $v = 0.2$). The measured average wall initial stiffness was thus 7% lower than the theoretical one, indeed a close correlation that confirms using ordinary flexural theory for estimation of the wall initial stiffness for walls with aspect ratios ($h_e/l_w$) of the order of 2 or higher.

7.6 COMPARISON WITH RCM WALL TESTING

This section compares the performance of the two Series 3 walls with reinforced concrete masonry (RCM) wall testing by Priestley and Elder [7-5]. Series 3 walls (S3-1 and S3-2) and the Priestley/Elder walls (PE1, PE2 and PE3) were of similar plane dimensions. The wall heights were comparable with 5.25 m for the S3 walls and 6.0 m for the PE walls. There were significant differences in the material properties and vertical reinforcement ratios. Nevertheless, meaningful comparison can be done after careful interpretation of results.

7.6.1 Priestley and Elder walls

The general dimensions and reinforcement of the walls tested by Priestley and Elder are shown in Fig. 7.22. The wall thickness was 140 mm and the wall length was 2400 mm. The wall height (top of foundation to point of lateral load application) was 6.0 m, thus 1.14 times that of the S3 walls. The longitudinal reinforcement at the wall base consisted of DH16 (16 mm deformed bar with nominal yield strength of 430 MPa) at 200 mm centres and was lap spliced at the wall base over 1000 mm for PE1 and PE2 and 1300 mm for PE3. Externally applied axial load and wall self-weight amounted to a total of 640 kN for PE1 and PE2, and 250 kN for PE3. PE2 had 600 mm long stainless steel confining plates embedded in the plastic hinge zone. It was estimated that the masonry crushing strength of the PE walls was approximately 25 MPa.

The primary intent with the PE wall testing was to evaluate the available ductility of tall (and slender) RCM walls as a function of axial load, lap splice length and bed joint confinement. The main conclusions from the PE tests were that the testing results gave reasonable confirmation of theoretical ductility capacity charts developed by the authors, however the theoretical predictions were found to be conservative for the lightly loaded wall (PE3). It was found that lap splicing in the plastic hinge zone clearly was a problem and should be avoided if possible.
It was confirmed that ductility capacity of RCM walls is more of a problem for tall walls than squat walls, but that the wall slenderness (height to thickness ratio) used in these tests was of little concern. It was finally concluded that the use of confining plates within the critical mortar joints of the potential plastic hinge will substantially improve response to severe seismic attack.

7.6.2 Strategy for comparison of results

In relation with evaluation of the present PCM wall testing it is interesting to draw parallels to the RCM wall performance discussed above. As suggested above this is not directly possible because of significant dissimilarities in wall height, masonry strength, applied axial load and design strength. The results of these differences are presented in Table 7.6 where the applied axial load, \( N \), the theoretical nominal strength, \( M_n = V_f h_w \), the experimental ultimate displacement capacity, \( \Delta_{u,exp} \) (average of both directions of testing) and drift capacity, \( \gamma_d \), are given. The axial load ratio due to self-weight is \( \xi_N \) and the wall gross section area is given as \( A_g = b_w l_w \). The theoretical strengths are based on [7-5] for the PE walls and Table 7.3 for the S3 walls.
The primary indicators of performance are:

1. Strength
2. Displacement capacity
3. Strength degradation
4. Energy dissipation
5. Damage/Repairability

Clearly items 1 and 2 above directly depend on applied axial load, \( N \), vertical reinforcement properties, masonry strength and wall dimensions. Therefore these performance indicators cannot be directly compared. Comparison of strength and displacement capacity consequently is based on extrapolation of the PCM wall results. This is done by application of the prediction procedure presented in Chapter 8 which was verified and calibrated against the Series 3 wall testing. Specifically, two predictions have been carried out for \( f'_m = 25 \) MPa, \( l_w = 2.4 \) m, \( b_w = 0.14 \) m, \( h_e = 6.0 \) m: (PCM-1) \( N = 640 \) kN and \( V_n = 272 \) kN which was achieved by a prestressing steel area of \( A_{ps} = 1115 \) mm\(^2\), and (PCM-2) \( N = 250 \) kN and \( V_n = 222 \) kN which was achieved by a prestressing steel area of \( A_{ps} = 1075 \) mm\(^2\). The initial prestress was taken as \( f'_{se} = 956 \) MPa or 63% of the yield strength, \( E_m = 20000 \) MPa, \( E_{ps} = 190 \) GPa, \( h_p = 0.456 m \), \( \varepsilon_{mu} = 0.013 \) and \( K = 1.06 \). Results of the prediction are presented in Table 7.7. Figs. 7.23 to 7.25 show the predictions overlaid on the experimental force-displacement curves by Priestley and Elder. It is noted that both PCM-1 and PCM-2 predictions were based on confined half height masonry blocks, CP100. Only PE2 had confinement plates embedded, in this case in full height masonry blocks, CP200. Thus comparisons and conclusions from Figs. 7.23 to 7.25

<table>
<thead>
<tr>
<th>Wall</th>
<th>N</th>
<th>( \xi N = N/(f'_m A_p) )</th>
<th>( V_n )</th>
<th>( M_n )</th>
<th>( \Delta u,exp )</th>
<th>( \gamma_{u,exp} )</th>
</tr>
</thead>
<tbody>
<tr>
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<td>273</td>
<td>1632</td>
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</tr>
<tr>
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</tr>
<tr>
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<th>( V_n )</th>
<th>( M_n )</th>
<th>( \Delta u,th )</th>
<th>( \gamma_{u,th} )</th>
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</tbody>
</table>

TABLE 7.6—Theoretical and experimental results for PE and S3 walls

TABLE 7.7—Theoretical performance of PCM walls for comparison
Fig. 7.23—PE1 experimental result and PCM-1 prediction envelope

Fig. 7.24—PE2 experimental result and PCM-1 prediction envelope
strictly only apply to the specific construction methods applied, and are only suggestive in terms of comparison of CP100 and CP200.

Items 3 through 5 can be discussed based on observations from the actual tests because the trends for these items are controlled by the nature of PCM or RCM construction rather than by actual applied force and strength.

### 7.6.3 Comparison of results

Comparing PE1 and PCM-1 of similar nominal strength in Fig. 7.23, it is found that the maximum strength of PE1 exceeded that of PCM-1. This is because of considerable strain hardening in the PE1 mild reinforcement. Only modest prestressing steel stress increase is expected for PCM-1 as a result of later deformation. It is also observed that the displacement capacity of PCM-1 is expected to exceed the displacement capacity measured for PE1.

Testing of PE2 with confining plates improved the RCM wall displacement capacity considerably. The maximum strength remained the same in comparison with PE1. Fig. 7.24 reveals that the expected displacement capacity of PCM-1 is lower than that measured for PE2. It is thus suggested that a RCM wall confined with CP200 and under high permanent axial load provides
higher displacement capacity than a PCM wall of similar proportions confined with CP100 or CP200. The actual measured drift capacity for PE2 of 1.28% may however be of little practical use because it may exceed the drift limitations set out in the structural codes.

Comparing the result from testing of PE3 with predictions for PCM-2 in Fig. 7.25, both walls subjected to relatively light permanent axial load, suggests that RCM walls provide larger displacement capacity than PCM walls, whether the RCM wall is confined or not. The ultimate drift ratios achieved by PE3 (test) and PCM-1 (prediction) of 1.48% and 1.19%, respectively, are high and could both be considered excessive in comparison with drift limitations given by structural codes.

The higher displacement capacity for RCM walls when compared to PCM walls is primarily caused by considerable difference in the vertical extent of the plastic hinge. It was found in the Priestley/Elder testing that the RCM plastic hinge typically extended vertically about 0.5l_w or approximately 1.2 m. PCM Series 3 testing suggested a vertical extent of the plastic deformation zone of about 0.08h_e or 0.48 m. Evidently higher displacement capacity can be expected from RCM walls for a given masonry extreme fibre maximum strain.

Comparing force-displacement histories for Series 3 walls (Fig. 7.9) and PE walls (Figs. 7.23 to 7.25) does not conclusively reveal whether strength degradation occurs more rapidly for one wall type than the other. Reinforced concrete masonry exhibits high hysteretic energy dissipation in comparison with PCM because of compression and tension straining of the flexural reinforcement. The prestressing steel in PCM walls dissipates little energy because little straining beyond the elastic limit as a result of unbonding of the prestressing is expected.

Perhaps the greatest behavioural difference between PCM and RCM is found in the category of Damage/Repairability. Fig. 7.11 shows the Series 3 PCM wall damage after nominal failure. As discussed in section 7.4.2, damage was limited to the lowest three masonry courses. There was no suggestion at any stage of testing that the ability to support vertical load was at risk. It is felt that damage could easily be repair, reinstating the full wall strength and integrity. On the contrary, substantial damage was sustained in the Priestley/Elder wall testing. After nominal failure, the first storey of all walls were cracked throughout, and face shell spalling and grout core damage was found in lowest 5-7 masonry courses (1.0 m to 1.4 m). It was suggested that the damage of the unconfined walls was impossible to repair while repair of the confined wall, PE2, possibly could be carried out successfully, despite the extent of damage and flexural
cracking. Clearly post-earthquake damage repair is much simpler for PCM because of the limited damage extent and virtually non-existing flexural cracking.

7.6.4 **Conclusions from comparison of PCM with RCM testing**

Conclusions of the PCM vs. RCM comparison can for masonry walls of similar dimensions, masonry material properties and strength be summarised in: (1) RCM walls are expected to exhibit larger displacement capacity than PCM walls because of longer plastic hinge length. (2) PCM walls are expected to sustain much less and more localised damage during strong ground motion than RCM walls, thus simpler and more economical post-earthquake repair. (3) Both PCM and RCM walls with confining plates are likely to exhibit drift capacities in excess of drift limitations for masonry walls given by structural codes.

It is reiterated that the above conclusions are based on experiments with significant differences in the material properties and structural dimensions, underlining the suggestive nature of the above comparison.

**7.7 CONCLUSIONS**

It is concluded that PCM walls of realistic proportions, strengthened in the flexural compression zones with confining plates, can successfully withstand severe cyclic loading imposed by an earthquake. Ductile response was measured with reliable drift capacity of 1.5%.

Only localised damage occurred, as shown in Fig. 7.11, making earthquake damage simple to repair. All visual damage occurred to the lowest 300-400 mm in the flexural compression zone. This is supported by the vertical strain plots shown in Figs. 7.15 and 7.16 and Table 7.5, that suggest inelastic masonry response in the lowest 400-600 mm of the walls.

Relatively little energy dissipation was observed during cycling of the walls.

Tendon force loss was not recorded.

Measurements at nominal flexural strength suggest an extreme masonry fibre strain in the order of 0.0010 to 0.0020, values substantially lower than 0.008 stipulated by NZS 4230:1990 [7-4] for confined masonry. At maximum wall strength, average strains of 0.0066-0.0080 were measured. At ultimate displacement, average measured strains in the extreme fibre were as
high as 0.019-0.024. At this stage the extreme masonry fibres no longer carried axial load as face shell spalling had occurred.

It was concluded that adequate shear friction between wall and foundation was provided by intentional roughening of the wall to foundation interface.

Comparison between PCM and RCM walls suggested that RCM walls are expected to exhibit larger displacement capacity than PCM walls because of longer plastic hinge length, that PCM walls are expected to sustain much less and more localised damage during strong ground motion than RCM walls, thus simpler and cheaper post-earthquake repair, and finally that both PCM and RCM walls with confining plates are likely to exhibit large drift capacities which may be in excess of drift limitations for masonry walls given by structural codes.
REFERENCES


Chapter 8

ANALYSIS AND DESIGN

8. PREDICTION OF WALL IN-PLANE BEHAVIOUR

This chapter considers the in-plane flexural response of post-tensioned concrete masonry cantilever walls with un-bonded prestressing tendons, where the lateral force is assumed to be acting at the top of the wall or at some effective height $h_e$. For other structural shapes and loading configurations, the formulae should be modified accordingly. Note that the term ‘tendon’ in the following refers to both prestressing strands and bars.

The present chapter is concerned with analytical methods for prediction of the in-plane force-displacement relationship for unbonded post-tensioned concrete masonry. The scope of such methods is to describe the wall behaviour throughout the entire loading range, including non-linear large displacement response that can be anticipated in a seismic event.

Two approaches of various complexities have been investigated:

(1) Flexure/Rocking model

(2) Finite Element model

Method (1) represents a simple approach based on ordinary flexural theory (plane sections) where integration of curvature along the wall height can be applied to determine the wall lateral displacement and vertical deformation. In the large displacement range a rigid body rocking analogy is used to describe the behaviour. This method largely focuses on establishing analytical expressions based on equilibrium and first engineering principles that can capture PCM response with reasonable accuracy. The prediction formulae can be used for limit state design as well as for establishing the most likely force-displacement relationship (continuous) for use in non-linear numerical analysis. This approach is the primary focus in this chapter.

Method (2): The Finite Element Model (FEM) technique supplements method (1) and is particularly useful for modelling of the wall internal force flow in the regions of force concentr-
tions, notably in the wall compression toe. The suitability of using FEM fibre elements for modelling of PCM in-plane response is investigated in Chapter 9 in conjunction with dynamic analysis.

A third method is the strut and tie methodology, with which the wall internal force flow can be analysed. This method is in particular useful for analysis of walls with openings and walls of unusual shape where the force flow cannot easily be described by method (1). As all walls investigated in this document were rectangular and without openings, pursuing this method for analysis was not considered merited in this document.

8.1 FLEXURAL RESPONSE OF CANTILEVER WALLS

It is assumed for flexural calculations that plane sections remain plane, i.e. there is a linear strain distribution across the wall length. This assumption enables, using first engineering principles, calculation of strength, stiffness and displacement, and implies distributed cracking up the wall height. From laboratory wall tests (Chapters 5, 6 and 7) it was observed that the PCM wall flexural response was primarily due to rocking where a crack opened at the base, and that distributed flexural cracking was non-existing. This type of rocking behaviour is attributed to prestressing with unbonded tendons. Despite this discrepancy between theory and observation, it appears that the assumption of plane section response with distributed wall cracks provides sufficiently accurate design rules.

8.1.1 Limit states

The flexural design procedure should be based on Limit State Design, as outlined by NZS 4203:1992 [8-1], which identifies two limit states, namely the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS). The serviceability limit state generally deals with function of the structure subjected to normal occupancy loads (service loads). It ensures durability and comfortable occupancy of the structure, by e.g. crack limitation or limitation of deflection. The criteria for the ultimate limit state relate to the strength and stability of a structure. This limit state is reached when the structure or a part of it reaches its ultimate capacity and fails to support further loading. These limit states generally need to be evaluated immediately after prestressing transfer and after long term losses (refer to Chapter 4).

Moreover, structures designed to withstand earthquake loading in a ductile manner should be designed according to the capacity design principle, an extension of ultimate limit state design.
The principle is based on identifying critical sections which will sustain inelastic deformation during a seismic attack. The strength of the critical sections are typically higher than the nominal capacity derived directly from codes and is termed the flexural overstrength. The rest of the structure is subsequently designed to withstand the forces arising in conjunction with development of the maximum credible strength (overstrength) at the critical locations.

**Serviceability Limit State:**

The flexural serviceability limit state for prestressed masonry is concerned with flexural strength, stiffness and deflections. The following flexural states represent the limiting flexural moments for a wall to remain elastic for un-cracked and cracked sections.

*First Cracking:* This limit state corresponds to the state when the extreme fibre of the wall decompresses (the tensile strength of concrete masonry is disregarded).

*Maximum Serviceability moment:* At this cracked section state, the compressive stress in the extreme compression fibre has reached its elastic limit set out by the code as a stress limitation. Reinforcement and concrete masonry remain elastic in this state.

**Ultimate Limit State:**

The flexural ultimate limit state for prestressed masonry is primarily concerned with flexural strength. Additionally for ductility purposes, overstrength, stiffness and deflections should be considered:

*Nominal strength:* The nominal strength according to NZS 4230:1990 [8-2] is per definition achieved when the concrete masonry strain in the extreme fibre, $\varepsilon_{mu}$, equals 0.0025 (unconfined concrete masonry). Notable, this value of $\varepsilon_{mu}$ may be altered in the future to provide greater alignment with other national masonry design codes (see Chapter 3).

*Overstrength:* This strength corresponds to the maximum moment strength developed by the wall, taking into account stress increase and yielding of the prestressing tendons. At this stage, large deformations are expected and the maximum concrete masonry strain is likely to have surpassed 0.0025. Past the maximum wall strength, the wall resistance gradually degrades until failure.

The applied forces and loads represented by the symbols $V^*$, $M^*$, $N^*$ and $P^*$ used in the following equations are all factored loads calculated according to the applicable limit state as defined.
in the New Zealand loading standard NZS 4203:1992. The axial force $N^*$ is due to dead and live loads, $P^*$ is the prestressing force (initial force after anchor lock-off or force after all long term losses), and $V^*$ is the applied lateral load due to lateral actions. It is assumed that moment $M^*$ only arises from lateral forces $V^*$, i.e. permanent loads and prestressing do not introduce permanent moment in the wall. It is also assumed that the prestressing tendons are placed symmetrically in the wall with respect to the wall centre line such that the resulting total prestressing force is applied at the wall centre line. Fig 8.1 shows the various definitions of wall dimensions and forces.

### 8.1.2 First cracking

The moment corresponding to first cracking $M_{cr}$ may be evaluated by Eqn. 8.1. The formula is based on the flexural state at which one wall end decompresses and the other end compresses to a stress of twice the average masonry stress $f_m$, as shown in Fig. 8.2.

$$M_{cr} = \frac{(P + N)l_w}{6} = f_m b_w l_w^2 \frac{l_w}{6} \quad \text{where} \quad f_m = \frac{(P + N)}{l_w b_w}$$

(8.1)

$$V_{cr} = \frac{M_{cr}}{h_e}$$

(8.2)

where $b_w$ is the wall thickness, $l_w$ is the wall length, $V_{cr}$ is the applied force at the top of the wall corresponding to the 1st cracking moment $M_{cr}$ and $h_e$ is the effective wall height (refer to Chapter 9 for definition of $h_e$).
The curvature at 1st cracking can be calculated as follows:

\[
\phi_{cr} = \frac{2(P + N)}{E_m l_w^2 b_w} = \frac{2f_m}{E_m l_w}
\]  

(8.3)

The lateral displacement of the top of the wall \(d_{cr}\) at \(V_{cr}\) should be based on the concrete masonry wall elastic properties (see Chapter 3) and consists of a component due to shear deformation \(d_{crsh}\) and a component due to flexure \(d_{crfl}\). Integration of the wall curvature is applied in order to evaluate the lateral displacement as shown schematically in Fig. 8.1.

\[
d_{cr} = d_{crfl} + d_{crsh} = \frac{2h^2}{3E_m l_w^2 b_w} + \frac{2(1 + v)(P + N)}{5E_m b_w}
\]  

(8.4)

where Poisson’s ratio typically is taken as \(v = 0.2\) for concrete materials [8-3]. It should be noted that the shear deformation component \(d_{crsh}\) can be of significant magnitude for squat walls under serviceability loads, whereas for the ultimate limit state it becomes increasingly insignificant.

### 8.1.3 Maximum serviceability moment

At the serviceability limit state (MSM), the applied lateral force has surpassed that necessary to initiate cracking at the base of the wall. The serviceability moment is limited by \(M_c\) which occurs when the stress in the extreme compression fibre at the base of the wall has reached
$k'f_m^*$ as shown in Fig. 8.3, where $f_m'$ is concrete masonry crushing strength. For prestressed concrete, $k$ (symbol adopted in this paper) is set out by NZS 3101:1995 [8-3], Table 16.1 and typically ranges between 0.45 and 0.55, dependent on load category. These values may also be assumed for prestressed concrete masonry.

It is noted that Eqn. 8.5 must be satisfied before use of the equations relating to the maximum serviceability moment. This requirement is generally fulfilled.

$$kf' > 2f_m$$  \hspace{1cm} (8.5)

It may be shown that the maximum serviceability moment can be calculated from force equilibrium as follows, adopting $k = 0.55$ from load category IV (infrequent transient loads):

$$M_e = \frac{f_m}{6} (3 - \frac{4f_m}{k'f_m}) l_w^2 b_w = f_m (0.5 - 1.21 \frac{f_m}{f_m'}) l_w^2 b_w = V_e h_e$$  \hspace{1cm} (8.6)

where $V_e$ is the corresponding lateral force.

$$M_e = (3 - \frac{4f_m}{k'f_m}) M_{cr}$$  \hspace{1cm} (8.7)

Eqn. 8.7 relates the MSM moment to the first cracking moment. The masonry is assumed to remain linear elastic, hence the extreme masonry strain $\varepsilon_{me}$ corresponding to $k'f_m$ can be found from:
The corresponding curvature at the wall base, $\phi_e$, is defined by:

$$\phi_e = \frac{\varepsilon_{me}}{c_e}$$

where

$$c_e = \frac{2(N + P)}{k f'_m b_w}$$

and

$$\frac{2 f_m l_w}{k f'_m} = \frac{3.6 f_m l_w}{f'_m}$$

$$\phi_e = \frac{(k f'_m)^2}{2 f_m E_m l_w} = 0.151 \frac{f_m^2}{f_m E_m l_w}$$

Fig. 8.4 shows the variation of moment and curvature along the height of the wall at the maximum serviceability moment, assuming plane section response. The curvature varies from $\phi_e$ at the base to $\phi_{cr}$ at the height, $h_{cr}$, at which the 1st cracking occurs. Between the heights $h_{cr}$ and $h_e$ the curvature varies linearly between $\phi_{cr}$ and zero. Fig. 8.3 and Eqn. 8.11 define the non-dimensional crack length at the base of the wall at the maximum serviceability moment, again assuming $k = 0.55$:

$$\gamma_e = 1 - \frac{2 f_m}{k f'_m} = 1 - \frac{3.6 f_m}{f'_m}$$

and Eqn. 8.12 defines the resulting cracked wall height.

$$h_{cr} = h_e \left( \frac{M_e - M_{cr}}{M_e} \right) = h_e \left( \frac{2 k f'_m - 4 f_m}{3 k f'_m - 4 f_m} \right)$$
The total displacement $d_e$ of the top of the wall due to flexure can then be calculated with:

$$d_e = d_{eff} + d_{esh} \quad (8.13)$$

The displacement $d_{eff}$ of the top of the wall due to flexure can then be calculated by double integration of curvature along the wall height (see Fig. 8.1) with the following result:

$$d_{eff} = \frac{2f_m h_{cr}}{E_m l_w \gamma_e} \left[ (h_e - h_{cr}) \left( \frac{\gamma_e}{1 - \gamma_e} \right) + h_{cr} \left( \frac{\gamma_e}{1 - \gamma_e} + \ln |1 - \gamma_e| \right) \right] + \frac{\phi_{cr} (h_e - h_{cr})^2}{3} \quad (8.14)$$

which may be approximated with, assuming $k = 0.55$:

$$d_{eff} = \left( 0.30 - 0.029 \frac{f_m}{f_m^*} h_e \frac{h_e^2}{E_m l_w} \right) \quad (8.15)$$

The shear deformation is defined by:

$$d_{esh} = \frac{12(1 + v) h_e}{5 E_m l_w b_w} V_e \quad (8.16)$$

At this flexural state, it is assumed that the relatively small deformations of the wall do not result in significant tendon force increase or migration of the tendon force eccentricity.

### 8.1.4 Nominal flexural strength

At this ultimate limit state, an equivalent rectangular stress block is assumed with a stress of $0.85f_m^* (\alpha = 0.85)$ and an extreme fibre strain of $\epsilon_{mu} = 0.0025$, corresponding to the definition of nominal strength in NZS 4230:1990 for unconfined concrete masonry. For confined masonry NZS 4230:1990 recommends using an average stress of $0.9Kf_m^* (\alpha = 0.9K$ with $f_m$ based on unconfined prism strength) and $\epsilon_{mu} = 0.008$. $K$ is defined in Eqn. 3.15. The corresponding moment $M_n$ and lateral force $V_f$ can be evaluated by simple equilibrium, as shown in Fig. 8.5, with the following equation:

$$M_n = (P + \Delta P) \left( \frac{l_w}{2} + \epsilon_f - \frac{a}{2} \right) + N \left( \frac{l_w}{2} - \frac{a}{2} \right) = V_f h_e \quad (8.17)$$

where $a$ is the length of the equivalent ultimate compression block given by:

$$a = \frac{P + \Delta P + N}{\alpha f_m^* b_w} \quad (8.18)$$
Fig. 8.5—Wall equilibrium at nominal flexural strength

In these equations, \( \Delta P \) accounts for the increase in tendon force that arises from the flexural deformation and \( e_t \) accounts for the associated tendon force eccentricity. Both \( \Delta P \) and \( e_t \) may initially be assumed to equal zero for simple use. This approach is similar to the method used in NZS 3101:1995. A better estimate of the nominal strength may be obtained from Eqn. 8.17 when taking into account the tendon force increase \( \Delta P \) and the associated tendon force eccentricity \( e_t \) (further on that issue below).

It is observed from Fig. 8.5 that there is moment reversal near the top of the wall due to \( e_t \) which results in reversal of curvature. This effect is not taken into account when calculating wall deformations in the following because it has a negligible effect on the predicted wall behaviour at nominal flexural strength.

The total lateral displacement \( d_n \) is given by the sum of the flexural displacement, \( d_{nfI} \), and shear displacement, \( d_{ns} \), corresponding to \( M_n \), and may be evaluated using Eqn. 8.19:

\[
d_n = d_{nfI} + d_{ns}
\]

where

\[
d_{nfI} = \text{(8.19)}
\]

Unconfined: \( d_{nfI} = (2.30\xi_n^2 - 1.38\xi_n + 0.856)\frac{f_m' h_e^2}{E_m' I_w} \) (8.20)

Confined: \( d_{nfI} = (7.63\xi_n^2 - 5.40\xi_n + 1.69)\frac{f_m' h_e^2}{E_m' I_w} \) (8.21)
Fig. 8.6—Wall deformation at nominal flexural strength

\[ d_{ash} = \frac{12(1 + \nu)h_e}{5E_m l_w b_w} \gamma_f \]  \hspace{1cm} (8.22)

\[ \xi_{en} = \frac{P + \Delta P + N}{f'_m l_w b_w} \]  \hspace{1cm} (8.23)

Eqns. 8.20 and 8.21 were developed using numerical integration and curve fitting, and are thus of an approximate nature, and are valid for axial load ratios, \( \xi_{en} \), of 0.05 to 0.025. The extreme fibre strain was taken as \( \varepsilon_{mu} = 0.003 \) for unconfined concrete masonry (consistent with international codes, but not NZS 4230) and 0.008 for confined concrete masonry. Detailed information on derivation of these equations may be found in Appendix A.

The total tendon force increase \( \Delta P \) at \( \varepsilon_{mu} \) of 0.003 (or 0.008) is difficult to evaluate for pre-stressed walls with unbonded tendons because the tendon stress increase depends on the deformation of the entire wall between points of anchorage. However, the force increase (or decrease) in each tendon in the wall cross section may be evaluated based on the estimated wall end elongation, \( u_e \), (tension end) and shortening (compression end), \( u_s \), assuming a linear variation of vertical deformation across the wall top as shown in Fig. 8.6. The following equations were established for unconfined and confined concrete masonry (refer to Appendix A):
Unconfined:  \[ u_e = (4.01\xi_n^2 - 2.37\xi_n + 0.835) \frac{f'_m h_e}{E_m} \]  
\[ u_s = (3.36\xi_n^2 - 2.12\xi_n - 0.073) \frac{f'_m h_e}{E_m} \]  

Confined:  \[ u_e = (22.5\xi_n^2 - 10.4\xi_n + 1.83) \frac{f'_m h_e}{E_m} \]  
\[ u_s = (1.67\xi_n^2 - 1.64\xi_n - 0.142) \frac{f'_m h_e}{E_m} \]  

In these equations, elongation is positive and shortening is negative. It is clear that the tendon force increase due to vertical deformation will increase the axial load ratio. Iteration using Eqns. 8.24 or 8.25 is therefore needed to find \( \Delta P = \sum \Delta P_j \) such that the calculated axial force ratio at nominal flexural strength, \( \xi_n \), injected in the equations on the right hand side in fact corresponds to the calculated tendon force increase on the left hand side of the equations.

The effective total tendon force eccentricity relative to the wall centre line can be evaluated by:

\[ e_t = \frac{\sum (P_j + \Delta P_j) y_j}{\sum (P_j + \Delta P_j)} \quad \text{where} \quad \Delta P_j = \frac{u_j A_{psj}}{l_j} E_{ps} \]  

\( P_j \) and \( \Delta P_j \) are the initial tendon force and tendon force increase of the j'th tendon, and \( y_j \) is the horizontal location of the j'th tendon with respect to the wall centre line taken as positive towards the tension end of the wall. The tendon vertical extension, \( u_j \), is defined in Fig. 8.6 and \( l_j \) is the tendon length (approximately the height of the wall \( h_{\text{w}} \), which is significantly longer than \( h_{\text{s}} \) for multi-storey building). \( A_{psj} \) is the area of the j'th tendon and \( E_{ps} \) is the elastic modulus of the prestressing steel. It must be ensured that \( P_j + \Delta P_j \) does not exceed the tendon yield strength.

Iteration process for calculation of \( \xi_n \) and \( d_n \):

1. calculate \( \xi_n \) using Eqn. 8.23 using \( \Delta P = 0 \).
2. calculate \( u_e \) and \( u_s \) using Eqns. 8.24 or 8.25.
3. calculate \( \Delta P = \sum \Delta P_j \) using Eqn. 8.26.
4. calculate \( \xi_n \) using Eqn. 8.23 using \( \Delta P \) from (3).
5. repeat steps (2) to (4) until convergence of \( \xi_n \).
6. calculate $M_n$ using Eqn. 8.17 and $d_n$ using Eqn. 8.19.

The masonry design codes BS 5628:1995 [8-4] and AS 3700:1998 [8-5] present formulae for calculating the tendon stress increase, but are not applicable for in-plane wall bending because they were developed for out-of-plane response. NZS 3101:1995 recognises that the design tendon force for unbonded tendons will exceed the tendon force following losses. Using the notation presented here, the increase in tendon force is given by:

$$\Delta P = A_{ps}\left(70\text{MPa} + \frac{f_m l_w b_w}{100 A_{ps}}\right)$$  \hspace{1cm} (8.27)

where $f_{se} = \frac{P}{A_{ps}}$, $f_{ps} < f_{py}$ and $f_{ps} < f_{se} + 400\text{MPa}$  \hspace{1cm} (8.28)

where $A_{ps}$ is the total prestressing tendon area, $f_{ps}$ is the resulting average tendon stress corresponding to $P+\Delta P$, $f_{py}$ is the tendon yield stress, defined in section 2, and $f_{se}$ is the tendon stress corresponding to $P$. This equation seems to provide reasonable results but has not been validated for all wall configurations. It would be prudent to assume a total tendon force increase of $\frac{1}{2}-\frac{3}{4}$ times the result calculated by Eqn. 8.27 when the prestressing tendons are approximately evenly distributed along the length of the wall. Eqn. 8.29 evaluates the resulting tendon eccentricity, $e_t$, due to the total tendon force increase $\Delta P$ acting at an eccentricity of $l_w/6$, assuming that tendons of equal area are evenly distributed across the wall.

$$e_t = \frac{l_w \Delta P}{6(P + \Delta P)}$$  \hspace{1cm} (8.29)

Having calculated $\Delta P$ and $e_t$, the nominal flexural strength, $M_n$, and corresponding displacement, $d_n$, can then be evaluated using Eqns. 8.17 and 8.19.

8.1.5 Yield strength

Contrary to reinforced concrete walls, the yield strength for unbonded prestressed walls is typically found at displacements beyond the displacement at nominal flexural strength. Structural testing has consistently shown that the behaviour of unbonded prestressed walls loaded beyond the nominal strength is dominated by rocking as illustrated in Fig. 8.7. Even for walls without specially placed confinement plates, experimental observations consistently demonstrate that the wall is able to support compression strains far beyond 0.003. In Fig. 8.7, the wall has rocked over by a displacement, $d_{ro}$, corresponding to a rotation $\theta$. At this state, it is assumed
that the extreme tendon at the tension side of the wall yields, resulting in a tendon strain increase of:

$$\Delta \varepsilon_{py} = \frac{(f_{py} - f_{ps})}{E_{ps}}$$

(8.30)

where $E_{ps}$ is the modulus of elasticity for the tendon steel as defined in Chapter 3 and $f_{ps}$ is taken as the tendon stress in the extreme tendon at nominal strength. If a wall is displaced laterally beyond $d_{ty}$, some reduction of prestress should be anticipated upon unloading. Notably, this does not mean that wall strength is permanently reduced because the tendons can be fully activated by subsequent loading excursions. The wall rotation $\theta$ can be related to the wall displacement increase at first tendon yield $d_{ty}$ and the tendon strain increase $\Delta \varepsilon_{py}$ in the following way:

$$\theta = \frac{\Delta \varepsilon_{py} h_e}{e_{te} - c} \quad \text{thus} \quad d_{ty} = \theta h_e = \frac{\Delta \varepsilon_{py} h_e^2}{e_{te} - c} = \frac{f_{py} - f_{ps}}{E_{ps}} \frac{h_e^2}{e_{te} - c}$$

(8.31)

where $a = \beta c$, and it is assumed $\beta = 0.85$ for unconfined masonry and $\beta = 0.96$ for confined masonry [8-2]. In this equation, $e_{te}$ is the eccentricity of the extreme tendon at the wall tension side with respect to the compressive end of the wall. The length of the compression zone, $c$, is calculated at the nominal strength based on Eqns. 8.18, thus assuming that the wall rocks about an axis at the distance, $c$, from the extreme compression fibre in the wall. As $d_{ty}$ is considered the displacement increment beyond $d_m$, the stress state in the extreme tendon should rigorously
be taken as $f_{ps}$, however using $f_{sc}$ (initial tendon stress in unloaded state) instead of $f_{ps}$ in Eqn. 8.31 generally result in little error.

Given $\theta$, the force increase in the individual tendons can be calculated as:

$$
\Delta P_{tyj} = \frac{\theta(e_{ij} - c)}{e_{te} - c} E_{ps} A_{psj} = (f_{py} - f_{ps}) A_{psj} \frac{e_{ij} - c}{e_{te} - c}
$$

(8.32)

$$
\Delta P_y = \sum \Delta P_{tyj}
$$

(8.33)

where $e_{ij}$ is the location of the j'th tendon with respect to the compression end of the wall, $A_{psj}$ is the area of the j'th tendon and $\Delta P_y$ is the total tendon force increase above that at $M_n$. Note that Eqn. 8.32 assumes linear variation of the tendon force increase with respect to the lateral location of the tendons. The resulting moment increase $M_{ty}$ is then given by:

$$
M_{ty} = \sum_{j=1}^{n} \Delta P_{tyj} \left( e_{ij} - \frac{a_y}{2} \right) = \sum_{j=1}^{n} \Delta P_{tyj} e_{ij} - \frac{a_y}{2} \Delta P_y
$$

(8.34)

where $n$ is the total number of tendons along the length of the wall and the compression zone length at first tendon yield may be calculated as:

$$
a_y = \frac{P + \Delta P_y + N}{\alpha f_m' b_w}
$$

(8.35)

Finally the yield moment $M_y$ and displacement $d_y$ can be evaluated as:

$$
M_y = (N + P + \Delta P) \left( \frac{l_w}{2} - \frac{a_y}{2} \right) + M_{ty} = V_y h_e
$$

(8.36)

$$
d_y = d_n + d_{ty}
$$

(8.37)

### 8.1.6 Flexural overstrength

The maximum credible strength of an unbonded prestressed wall may be evaluated by assuming that all tendons have reached their yield strength. Consequently, the flexural overstrength, $M_o$, maybe evaluated as:

$$
M_o = (N + P_y) \left( \frac{l_w}{2} - \frac{a_o}{2} \right) = V_o h_e
$$

(8.38)

where $a_o$ is the length of the equivalent ultimate compression block and $P_y$ is the total tendon force when all tendons are yielding given by:
\[ a_o = \frac{N + P_y}{\alpha f_{m}^c b_w} \quad \text{and} \quad P_y = A_{ps} f_{py} \]  

(8.39)

At this state, it is assumed that the tendon closest to the flexural compression zone has reached its yield stress. The resulting displacement can then be evaluated using the following equation which is similar to Eqn. 8.31:

\[ d_o = d_n + \frac{f_{py} - f_{ps}}{E_{ps}} \frac{h_e^2}{e_{tc} - a_o/\beta} \]  

(8.40)

In this equation \( e_{tc} \) is the distance from the compression end of the wall to the closest tendon and \( f_{ps} \) is the tendon stress in the same tendon at nominal strength.

It is noted that the Eqn. 8.40 is not appropriate if the closest tendon is located within the flexural compression zone, i.e. \( e_{tc} < c \), and that if the tendon closest to the compression zone is near to the location of the flexural neutral axis, unrealistically large values of \( d_o \) are calculated. When all tendons are located near the wall centreline, the wall yield strength coincides with the wall overstrength. It can be argued for conservatism that the tendon yield stress, \( f_{py} \), in Eqn. 8.39 should be replaced with the tendon ultimate strength, \( f_{pu} \), in order to establish the maximum credible wall flexural strength. It is, however, unnecessary to modify Eqn. 8.40 accordingly because the tendon strain at ultimate strength is of the order of 5% and therefore not attainable in reality for walls of any geometry. Use of \( f_{pu} \) is only necessary if the prestressing is unbonded over a height significantly shorter than the wall height.

### 8.1.7 Ultimate displacement capacity

The ultimate displacement is limited by the strain capacity of the tendons as well as the crushing strain of the masonry. Generally, the tendon ultimate strain is of the order of 5%, which would require unrealistically high lateral displacement to govern. As a result concrete masonry compression failure is expected. Confinement by the foundation is likely to increase the failure masonry strain beyond 0.003. As the extreme concrete masonry fibres fail, there is a tendency for the compression zone to migrate towards the centre of the wall, reducing the wall strength gradually. Experiments conducted at the University of Auckland and presented in Chapters 5, 6 and 7 have shown drift ratio capacities of 1% - 2% for prestressed concrete masonry walls of various aspect ratios, suggesting high displacement capacity. It is noted that this limit state may occur before tendon yielding, depending on the wall aspect ratio, the prestressing steel area and the initial tendon stress \( f_{se} \).
The drift ratio or the drift angle is in this document defined as the ultimate displacement $d_u$ divided by the effective height, and is thus termed ‘drift capacity’:

$$\gamma_u = \frac{d_u}{h_e} \quad \text{(8.41)}$$

Evaluation of the extreme masonry strain at displacements beyond nominal flexural strength necessitates definition of a plastic hinging zone at the bottom of the wall. Assuming that all lateral displacement at the top of the wall is due to rotation, $\theta$, of the plastic hinge as shown in Fig. 8.8, the masonry extreme fibre strain, $\varepsilon_{mu}$, can be related to the wall lateral displacement, $d_u$:

$$d_u = \theta \left( h_e - \frac{h_p}{2} \right) \quad \text{and} \quad \theta = \phi \frac{h_p}{c} = \frac{\varepsilon_{mu} h_p}{c} \quad \text{(8.42)}$$

$$d_u = \frac{h_p \left( h_e - \frac{h_p}{2} \right)}{c} \varepsilon_{mu} \quad \text{where} \quad c = \frac{a}{\beta} = \frac{P + \Delta P + N}{\alpha f_m^b b_w \beta} = \frac{\varepsilon_{u} l_w}{\alpha \beta} \quad \text{(8.43)}$$

In this equation, $\Delta P$ should correspond to the actual tendon stress state at the displacement $d_u$. For consistency with the calculations at $M_n$, $M_y$ and $M_o$, it is assumed $\alpha = 0.85$ and $\beta = 0.85$ for unconfined concrete masonry (U200) and $\alpha = 0.9$ and $\beta = 0.96$ for confined masonry (CP200 and CP100). Refer to Chapter 3 for definition of U200, CP200 and CP100. It is accentuated that Eqn. 8.43 is of idealised nature and simply attempts to relate the lateral displacement to the masonry strain state in the compression toe region at the wall state where initiation of strength degradation due to masonry crushing is anticipated to commence. Eqn. 8.42 assumes that the
TABLE 8.1—Basic plastic deformation zone parameters

<table>
<thead>
<tr>
<th>Concrete Masonry</th>
<th>$V_{max}$</th>
<th>$0.8V_{max}$</th>
<th>$V_{max}$ and $0.8V_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\varepsilon_{mu}$</td>
<td>$\varepsilon_{mu}$</td>
<td>$h_p/h_e$</td>
</tr>
<tr>
<td>U200**</td>
<td>0.016**</td>
<td>0.020*</td>
<td>0.071*</td>
</tr>
<tr>
<td>CP200**</td>
<td>0.016**</td>
<td>0.020*</td>
<td>0.071*</td>
</tr>
<tr>
<td>CP100**</td>
<td>0.008</td>
<td>0.013</td>
<td>0.076</td>
</tr>
</tbody>
</table>

* calculated using Eqn. 8.43, * based on visual inspection, ** refer to Chapter 3

Total rotation $\theta$ occurs at a height of $h_p/2$ above the wall base. This is in line with the current thinking for plastic hinge zone rotation for reinforced concrete masonry walls [8-6]. For evaluation of $d_u$, it is acceptable to interpolate between the axial forces calculated at nominal flexural strength, first tendon yield and overstrength relative to the displacements $d_n$, $d_y$ and $d_o$, as applicable (with a maximum of $N+P_y$). The base shear corresponding to $d_o$ can be based on Eqn. 8.17 using the appropriate axial force or on interpolation between $V_o$, $V_y$ and $V_o$ with a maximum of $V_o$.

The approach used in this document for calculating maximum displacement capacity is parallel to that used for reinforced concrete masonry (RCM). Eqn. 8.43 relates the total displacement to the total strain in the plastic deformation zone, and disregards any elastic deformation of the wall above the plastic deformation zone. For RCM, the displacement due to plastic hinging constitutes a displacement increment $d_p$ due to a plastic strain increment of $\varepsilon_{mp}$ where $d_u$ is calculated as $d_y + d_p$ and $\varepsilon_{mp}$ as $\varepsilon_{mu} - \varepsilon_{my}$ [8-6]. The quantities $d_y$ and $\varepsilon_{my}$ represent the wall displacement and extreme fibre strain at first yielding of the extreme mild steel reinforcing bar. The reason for using the simple approximate relationship defined by Eqn. 8.43 is that the limit state defined by $d_y$ and $\varepsilon_{my}$ does not exist for PCM (in the conventional sense) and that $\varepsilon_{mu}$ for PCM proves to be very large relative to typical values of $\varepsilon_y$ (normally elastic concrete masonry strain) for RCM.

The ultimate masonry strain capacity, $\varepsilon_{mu}$, is associated with flexural action and exceeds by far the unaxial strain capacity determined by prism testing (refer to Chapter 3). Experimentally determined values of $\varepsilon_{mu}$ and $h_p/h_e$ are given in Table 8.1 and were based on a compilation of results from the structural testing of unbonded post-tensioned concrete masonry walls described in Chapters 5 through 7. Given the small number of tests and unknown repeatability of these, it would be prudent to reduce the strain values given in Table 8.1 for design. The
design value of the ultimate strain capacity should be taken as $\varepsilon_{mu} = \varepsilon_{\mu}/\gamma$, where $\gamma$ is taken as 1.3 to 1.5.

Normalisation of $h_p$ with $h_c$ was based on comparison of results from the three test series which indicated that $h_p$ was likely to be related to the effective wall height, $h_e$, rather than the wall length, $l_w$. The values of $h_p$ for U200 and CP200 (wall test Series 1 and 2) were based on visual inspection of the wall toe regions when strength degradation of $0.8V_{max}$ had occurred. Inelastic strain ($\varepsilon_m > 0.0015$) in the masonry above the first course (200 mm) was not measured in any of the Series 1 and 2 U200 and CP200 tests, thus $h_p = 200$ mm was assumed. Series 1 and 2 testing results do not confirm that U200 or CP200 concrete masonry are capable of developing inelastic strain above the first course, therefore the results for U200 and CP200 should for taller walls be extrapolated with caution.

The plastic deformation zone length for CP100 masonry was based on the results from testing Series 3, which reflected realistic wall dimensions in multi-storey buildings. Extensive and reliable measurement readings revealed that inelastic strain ($\varepsilon_m > 0.0015$) occurred over a height of 400-600 mm at both $V_{max}$ and $0.8V_{max}$. This confirms that CP100 is capable of developing inelastic strain over a height of four or more courses of concrete masonry. The longer $h_p$ for the 5.25 m high Series 3 walls also explains why the average strain (theoretical) in the plastic deformation zone for $V_{max}$, given in Table 8.1, for CP100 is lower than that of CP200. The $h_p$ value given in Table 8.1 for CP100 was based on actual measured values, while $h_p$ values given in Table 8.1 for CP200 were based on visual inspection. For design purposes $h_p/h_c$ for CP100 may be taken as 0.08 which conforms well with that typically used for reinforced concrete walls [8-6].

It may seem counter-intuitive that the values for $\varepsilon_{\mu}$ given in Table 8.1 for U200 and CP200 are higher than those for CP100. However as suggested above, the results for U200 and CP200 walls cannot directly be compared with the results for CP100 walls because the modular nature of the masonry blocks determined that $h_p = 200$ mm for the squat walls U200 and CP200.

It was found by structural testing when using CP200 and CP100, that the wall strength persisted at levels close to $V_{max}$ for displacements beyond $d_{v_{max}}$ (corresponding to $V_{max}$), until displacements near $d_{0.8v_{max}}$ (corresponding to $0.8V_{max}$) were reached. It can therefore be argued that dependable wall displacement capacity, $d_{\mu}$, with little strength degradation may be calculated using $\varepsilon_{\mu}$ related to $0.8V_{max}$. This is generally not the case for U200, for which testing
proved that significant strength degradation was imminent after $d_{\text{max}}$. Measurements relating to $0.8V_{\text{max}}$ can be used for U200 if it is assumed that reliable wall performance persists until the strength had degraded to $0.8V_{\text{max}}$.

It appears from Table 8.1 that the plastic zone heights are of the order of 7-8% of the wall height, or 200 mm to 400 mm for the tested wall configurations. This is a short length when compared with typical plastic hinge lengths for reinforced concrete masonry walls of the order of $0.5l_w$ [8-6], or 900 mm to 1,500 mm when applied to the experimental wall lengths. Embedding of confining plates can consequently be limited to a small region in each end of the wall, say over a vertical distance of 1.5 to 2 times $h_p$ and a horizontal extent of 1.5 times $\xi l_w$.

Vertical strain measurements from testing Series 1, 2 and 3, as discussed in Chapters 5, 6 and 7, confirm the magnitude of strain and plastic zone length proposed in Table 8.1. Nevertheless, it is emphasised that the results in Table 8.1 are of preliminary nature and should be used with caution due to considerable scatter of results, and as stated above, that these values are of theoretical and empirical character. Additionally, the proposed ultimate strains should not be regarded as the reliable ultimate uniaxial strain capacity of concrete masonry.

### 8.1.8 Calculation example

Consider the wall shown in Fig. 8.9(a). It is assumed that the five storey wall is 15 m high, 3.6 m long, 190 mm thick and prestressed with five 15.2 mm high strength prestressing strands ($A_{psj} = 140$ mm$^2$). Half height concrete masonry blocks from the 20 series (100 mm high) are used in the plastic deformation zone; regular 20 series blocks elsewhere. The wall self weight is calculated to be 225 kN and the dead load of the floors and roof amounts to 0.5 MPa at the base of the wall, resulting in a total dead load of 567 kN.

The calculation is performed on the equivalent structure shown in Fig. 8.9(b) with an effective height, $h_e = 2/3 \times h_w = 10$ m (refer to Chapter 9 for further considerations on $h_e$), at which the wall lateral load is assumed to be applied. The tendons are placed symmetrically about the wall centre line at zero, ±200 mm and ±400 mm eccentricities from the wall centre line (the five strands are represented with one line in Fig. 8.9). In the calculation, the tendon elastic elongation capacity is based on the actual tendon length, approximated with $h_w$, using the effective tendon elastic modulus of $E_{ps} \times h_e \times h_w$. An initial tendon stress of $0.67f_{pu}$ is selected, based on an estimated first tendon yield at a lateral drift of about 1.5% assuming that the wall rocks as a
rigid body around the lower corners. A total prestressing force of 831 kN is found, resulting in an initial axial load ratio of $\xi = 0.114$.

Confinement plates are imagined embedded in the horizontal bed joints in the wall corners by the base over a height of $2 \times h_p = 2 \times 0.076 \times 10$ m = 1.5 m and $K = 1.08$ is assumed (refer to Chapter 3). The confinement plate length is taken as $2 \times \xi l_w$ or about 800 mm. It is assumed that the height of the plastic hinge zone is $0.076 \times h_c = 0.76$ m and the ultimate flexural strain is 0.013, as recommended in Table 8.1.

Table 8.2 and Fig. 8.10 present the predicted wall in-plane response with the base shear $V$, lateral displacement $d$ and tendon force increase $\Delta P$ related to the equivalent structure shown in Fig 8.9(b). Material properties and wall dimensions are specified in Fig. 8.10. Specific details on the calculation example may be found in Appendix B. It is seen in Fig. 8.10 that wall sof-
tensioning initiates between the maximum serviceability moment and the nominal strength limit states. Between the two formerly mentioned limit states, the theoretical crack length at the wall base increased from 0.60×l_w to 0.88×l_w. The wall ultimate displacement capacity is reached 34 mm after the first tendon yield limit state. It is expected that the tendons remain elastic up to a drift of 1.58% (displacement of 158 mm) which is slightly higher than the initially considered drift of 1.5% before tendon yield. The displacement at wall overstrength is, in this case, only of theoretical interest. For this particular wall, the ratio of overstrength to nominal strength amounted to 1.11.

8.2 DESIGN CONSIDERATIONS

8.2.1 Required prestressing force and area

The required prestressing force, P, may be determined by the applied actions at either the Serviceability Limit State (SLS) or at the Ultimate Limit State (ULS). For prestressed concrete with bonded tendons, it is normally SLS that governs the required prestressing force, however for prestressed concrete masonry with unbonded tendons, it is not clear whether SLS or ULS loads govern. This is largely because the wall ultimate strength (nominal flexural strength) does not benefit from significant tendon stress increase due to flexural deformation. It appears for PCM walls with unbonded tendons that the dependable nominal strength Φ_Mn rarely is more than 20% larger than the maximum allowable elastic service moment, M_ec, given by Eqn. 8.6.
Supposing that for wind actions the ultimate flexural load $M_u^*$ is roughly equal to 1.4 times the service load $M_s^*$, it is found that the ULS requirement is likely to govern the design axial prestress. For seismic loads determined for limited ductility demands, the strength requirement at ULS again appears to govern the design axial prestress. Current studies indicate that the ULS requirement given by Eqn. 8.44 generally is satisfied, if the required design axial prestress has been determined based on un-cracked section analysis at SLS in accordance with Eqn. 8.45.

$$\phi_f M_n \geq M_u^*$$  \hspace{1cm} (8.44)

$$M_{cr} \geq M_s^*$$  \hspace{1cm} (8.45)

Prior to determination of the required prestressing force, $P$, it needs to be decided whether the wall must remain un-cracked at SLS or not. In that regard it is reiterated, that only a single crack at the base is expected to open when the 1st cracking moment is exceeded at the base, and that this crack will close immediately upon unloading of the wall, unlike the behaviour for conventionally reinforced concrete construction. Consequently PCM wall cracking at SLS loads is not a durability issue but rather an issue related to function of the structure. Seen on the basis of the expected crack pattern, it could be argued to require that PCM walls remain un-cracked for all SLS load combinations, except for earthquake load combinations.

Alternatively, the required prestressing force could be determined by shear strength considerations at ULS.

When the required prestressing force $P$ has been determined, the prestressing steel amount is calculated, assuming that $P$ needs to be present after all losses (refer to Chapter 4), and at the same time respecting the allowable prestressing steel stress before and immediately after prestress transfer.

The anchorage region, to which the concentrated prestressing load is applied, shall be designed according to NZS 3101:1995, section 16.3.9, in order to avoid bursting, splitting and spalling.

It is conceivable that the prestressing force needs to be applied in two stages, depending on the early strength of the masonry and the magnitude of the anticipated prestressing loss. Initially, a lower prestressing force could be applied to resist construction loads. The final prestressing force is then applied at a later date.
8.2.2 Un-cracked section analysis

To ensure un-cracked response at SLS Eqn 8.45 needs to be satisfied. The required prestressing force is then given by:

\[ P \geq \frac{6M_s^*}{l_w} - N_s^* \]  

(8.46)

8.2.3 Cracked section analysis

In this case, the serviceability moment may be equated with the maximum elastic moment (see Eqn. 8.6), resulting in the required average prestress on the wall being given by:

(a) \[ M_s^* \leq \frac{f_m}{6} \left( 3 - \frac{4f_m}{k_{f_m'}} \right) l_w^2 b_w \]

(b) \[ \frac{M_s^*}{f_m' l_w^2 b_w} \leq \frac{\xi}{6} \left( 3 - \frac{4\xi}{k} \right) \] where (c) \( f_m = \xi f_m' \)

(8.47)

Plotting the left hand side of Eqn. 8.47(b) for realistic values of \( \xi \) between 0 and 0.4 and values of \( k \) between 0.4 and 0.6, the curves shown in Fig. 8.11 materialise. Given the right hand side of Eqn. 8.47(b) and the allowable value of \( k \), \( \xi \) may be determined graphically. The average axial concrete masonry stress \( f_m \) may then be determined by Eqn. 8.47(c) and the required prestress force, \( P \), may be determined by:
\[ P \geq f_m l_w b_w - N_s^* \] (8.48)

8.2.4 Ultimate limit state

8.2.4.1 Strength design

As the ultimate limit state flexural actions may govern the design prestressing force \( P \), it must be verified that the dependable flexural strength exceeds the ultimate flexural actions as defined by Eqn. 8.44, using \( \phi_t = 0.85 \).

This can be done by evaluating the required prestressing force, \( P \), which easily can be derived by trial-and-error from the equations presented in section 8.1.4 by initially assuming that \( \Delta P = 0 \). First approximation of the required total prestressing steel area \( A_{ps} \) may then be calculated with the assumption that the effective prestressing steel stress after all losses, \( f_{se} \), amounts to about \( 0.50f_{pu} \) (refer to Chapter 4). A closer approximation can then be obtained when \( \Delta P \) (Eqns. 8.24 or 8.25) is included in the calculation of the nominal strength \( M_n \). Depending on the required accuracy, \( P \) can be determined after several iterations.

8.2.4.2 Capacity design

Calculation of the wall overstrength, \( M_o \), is of considerable significance for ductile seismic design. The principle of capacity design states that the forces generated throughout the structure, caused by development of the maximum strength of the ductile yielding components creating a mechanism, must be exceeded by the dependable capacity of all other joining components. This will ensure, in the event of a major earthquake, that yielding will occur at the specially selected locations (plastic hinges) and that all other types of potential failure modes are eliminated. This means for a cantilever wall, that the wall shear strength, \( V_p \), must exceed the base shear, \( V_o \), that develops the maximum flexural strength of the wall (overstrength \( M_o \)) and that the wall flexural strength outside the plastic hinge must exceed the wall moment distribution corresponding to development of the wall overstrength, \( M_o \). Additionally, the moment distribution (demand) arising from development of flexural overstrength may have to be amplified to take into account higher mode effects. Refer to Paulay and Priestley [8-6] for further discussion.
8.2.4.3 Displacement capacity

For ductile seismic design, it is necessary to ensure that the nominal flexural strength and yield strength are sufficiently large to limit excessive displacements (storey drift) during seismic attack. This concept is similar to reducing the ductility demand for reinforced concrete masonry by increasing the flexural strength (yield strength). A prestressing steel area increase is needed if the wall strength is insufficient to ensure reasonable limitation to the lateral displacements.

It is desirable to ensure that the prestressing steel in an unbonded post-tensioned concrete masonry wall responds elastically to seismic excitation for several reasons: the wall retains (1) the initial stiffness, (2) the strength, (3) the self centring capability, and (4) restressing of the tendons after the seismic event is not necessary. A low initial tendon stress relative to the tendon yield strength allows for larger wall displacements before first tendon yield at the displacement of \( d_p \). Conversely, a relatively high initial tendon stress is desirable because it reduces time dependent prestress loss and reduces the required prestressing steel area (better economy). Ideally, the seismic displacement demand, \( d_b^* \), should be exceeded by both the tendon yield displacement, \( d_p \), and the displacement capacity, \( d_u \).

The displacement demand imposed on an unbonded wall structure in a seismic event, \( d_b^* \), cannot be predicted using existing loadings codes and acceleration spectra. However, using numerical non-linear time-history analysis, the displacement demand can be estimated as described in Chapter 9.

8.2.5 Base sliding

Base sliding response should be avoided, however such behaviour is not anticipated for prestressed concrete masonry walls (PCM) of normal wall aspect ratios and prestressing levels. Base sliding for PCM is controlled by the available sliding resistance, which is provided by shear friction by means of aggregate interlock mechanisms. A check at ultimate using the following formula is recommended:

\[
V_{base} \leq \phi \mu (P + N) \tag{8.49}
\]

where \( V_{base} \) is the base shear due to lateral forces, \( \mu \) is the dependable coefficient of friction for concrete to concrete contact and \( \phi \) is the appropriate strength reduction factor generally taken as 0.85, except when \( V_{base} \) is derived from capacity design considerations where \( \phi = 1.0 \). The
recommended value of the shear friction coefficient (Coulomb friction), $\mu$, depends on the concrete roughness amplitude (average distance between ‘valley’ and ‘peak’), $r_s$. The following values are applicable to concrete [8-6] but may be used for concrete masonry as no values specifically for concrete masonry are available:

\[
\begin{align*}
(a) & \quad \mu = 1.4 \quad \text{for} \quad r_s \geq 5 \text{ mm} \\
(b) & \quad \mu = 1.0 \quad \text{for} \quad 2 \text{ mm} \leq r_s \leq 5 \text{ mm} \\
(c) & \quad \mu = 0.7 \quad \text{otherwise}
\end{align*}
\]

Eqns. 8.50 (a) and (b) both relate to concrete surfaces that have purposely been roughened, and all equations (a, b and c) are based on concrete placed against a clean surface.

For concrete masonry walls, the foundation and above floor slabs (whether intentionally roughened or not) must be clean before the first course of concrete blocks is laid. ‘Wash-outs’ (clean-out ports) may be necessary for grouted construction to ensure that mortar dropping inside the flues does not prohibit effective aggregate interlock between grout and the foundation/floor slab. New Zealand standard grout with specified strength of 17.5 MPa typically contains 7 mm aggregate. According to Collins [8-7], aggregate interlock increases with increasing aggregate size. Therefore the fine aggregate of the grout is unfavourable to larger aggregate in concrete. Furthermore, it should be kept in mind that the largest particle type in mortar is sand and that the mortar bed joints typically occupy 25-50% of the wall cross section. It can be argued that the maximum friction factor of 1.4 given in Eqn. 8.50, in reality, is not achievable. The minimum value of $\mu = 0.7$ was violated by some of the later tests in Series 1 and 2. This is of little concern because base sliding for these walls was highly affected by multiple use of the same foundation. The foundation top surface had after some tests effectively been smoothened to a very small roughness amplitude that would not apply to real construction.

The propensity for sliding is highly dependent on the wall aspect ratio: $h_c/l_w$. A reduction of aspect ratio results in an increase of the base shear needed to develop the wall flexural strength. A high aspect ratio is therefore preferred in terms of reducing the propensity for base sliding. The minimum aspect ratio permitted can be estimated in the following way. Consider the nominal flexural strength of a cantilever wall assuming no tendon force increase:

\[
M_n = (N + P)\left(\frac{l_w}{2} - \frac{a}{2}\right) = V_f h_c
\]

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where

\[ a = \frac{N + P}{\alpha f_m' b_w} = \xi \frac{l_w}{\alpha} \quad \text{and} \quad \xi = \frac{N + P}{f_m' l_w b_w} \]

The maximum base shear can be expressed by:

\[ V_{max} = 1.33 V_f \omega_v \]  \hspace{1cm} (8.51)

where the 1.33 factor accounts for maximum conceivable wall strength (overstrength) and \( \omega_v \), the seismic dynamic amplification factor, accounting for base shear amplification due to higher mode effects (see section 9.3.2.4). Substitution of the equation for \( a \) into the equation for \( M_n \) and solving for \( V_f \) gives:

\[ V_f = \frac{1}{2} (N + P) \frac{l_w}{h_e} \left( 1 - \frac{\xi}{\alpha} \right) \]  \hspace{1cm} (8.52)

Injecting Eqn. 8.52 into Eqn. 8.51 results in the following requirement for the wall aspect ratio:

\[ \frac{h_e}{l_w} \geq \frac{0.67 \left( 1 - \frac{\xi}{\alpha} \right) \omega_v}{\phi \mu} \]  \hspace{1cm} (8.53)

A suggested minimum aspect ratio can be estimated using \( \xi = 0.1 \), \( \alpha = 0.96 \), \( \omega_v = 2.5 \), \( \phi = 1.0 \) and \( \mu = 0.7 \). Using these values in Eqn. 8.53, a minimum wall aspect ratio of approximately 2.1 is found. Assuming that the effective wall height amounts to \( \frac{2}{3} \) of the actual wall height, a minimum absolute aspect ratio of \( h_w/l_w = 3.2 \) is found. The value of \( \omega_v = 2.5 \) used above originated from section 9.3.2.4. A more detailed calculation reflecting the actual wall dimensions, overstrength, roughness amplitude and base shear amplification may justify a lower aspect ratio.

It should be kept in mind that there is always an effective concrete masonry compressive force at the base of the wall (in excess of \( N \)) due to flexure, even after initiation of yielding of the tendons. This differs from the typical response of conventionally reinforced walls cyclically loaded beyond yield, where the wall effectively is supported laterally at the base by dowel action alone. Finally, if significant degradation of the concrete masonry in the wall corners is expected to occur, other means of inhibiting base sliding may be required, such as dowel bars, sliding blocking devices or recessing of the wall into the foundation.
8.3 VERIFICATION OF PREDICTION METHOD

Three structural testing series were carried out in the Civil Test Hall at the University of Auckland. Test Series 1 focused on single storey walls built with ordinary concrete masonry (unconfined) and is described in Chapter 5. Test Series 2, reported in Chapter 6, focused on single storey walls with enhanced wall corners such that the masonry strength and vertical strain capacity was increased, with the overall scope of increasing the wall displacement capacity. Test Series 3 is described in Chapter 7 and deals with testing of two three-storey walls of realistic proportion. This testing series consisted of two walls.

A detailed comparison between predictions based on the flexure/rocking model and experimentally obtained results may be found in Appendix C. The following conclusions were drawn:

**Initial stiffness:**
For walls of low aspect ratio (say below 1.0), the predicted initial stiffness based on $V_{cr}$ and $d_{cr}$ overestimates the actual wall stiffness by a factor of the order of 1.5. For aspect ratios above 2, accurate prediction is expected.

**Global response:**
Using the predicted base shear and displacement at maximum serviceability moment, at nominal strength, at first tendon yield and at overstrength, accurate and consistent estimation of the force-displacement envelope (skeleton curve) can be achieved.

**Displacement capacity:**
The predicted displacement capacity, $d_{in}$, was captured with reasonable accuracy using the values for extreme fibre strain and plastic zone length suggested in Table 8.1. Based on structural testing of 2.8 m high walls, $\varepsilon_{mu} = 0.020$ and $h_p/h_e = 0.071$ are adopted for unconfined and confined masonry of 200 mm block height (U200 and CP200). Predictions for the walls using confined masonry of 100 mm block height (CP100) were based results from testing of 5.25 m high walls: $\varepsilon_{mu} = 0.013$ and $h_p/h_e = 0.076$. The strength associated with $d_u$ for confined concrete masonry (CP200/CP100) should be based on interpolation of base shear between the nominal strength, first yield strength and overstrength limit states, as a function of displacement, as applicable. The maximum strength achieved at $d_u$ should be limited by $M_u$. For unconfined concrete masonry it appears prudent to calculate the wall capacity at $d_u$ using $0.8V_u$. 

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8.4 REFERENCES


Chapter 9

DYNAMIC ANALYSIS

9. INTRODUCTION

This chapter is concerned with non-linear time-history analysis of unbonded post-tensioned concrete masonry (PCM) cantilever walls. Analyses were carried out in order to clarify the relationship between wall stiffness and strength, and the wall displacement demand due to dynamic seismic excitation. The effects of the PCM wall force-displacement characteristics and structural damping were investigated for a selected building period range. Results from multi-degree of freedom FEM modelling were compared with results from single degree of freedom equivalent structure modelling. Pseudo displacement spectra ($S_D$) were developed for design use, and a displacement focused design procedure is outlined.

Following research presented in the previous chapters, it can be argued that the hysteretic form of PCM for in-plane static loading is well established. However, the dynamic response of such walls, when subjected to seismic excitation, is uncertain and remains a controversial issue because many researchers are concerned that the inherent low energy dissipation associated with unbonded tendons may lead to excessive displacement demand.

9.1 MULTI-DEGREE OF FREEDOM STRUCTURE

This section defines a prototype structure and outlines appropriate modelling using the finite element method (FEM).

9.1.1 Prototype structures

Fig. 9.1(a) shows a typical wall suitable for a five-storey apartment/office building. The wall is 3.6 m long, 15 m high (3 m storey height) and 190 mm thick. It is assumed that the lateral forces arising from ground shaking are distributed evenly between the walls carrying the lateral force in a given direction, and that there is negligible coupling between these walls because of
Fig. 9.1—5-Storey PCM wall prototype and modelling

relatively flexible floor slabs. Thus the building response is represented by a wall analysed as a free-standing cantilever. The wall is post-tensioned with six 15.2 mm high strength prestressing strands (elastic modulus: $E_{ps} = 190$ GPa, single tendon area: $A_{ps} = 140$ mm$^2$, ultimate strength: $f_{pu} = 1785$ MPa) located in pairs at -800 mm, zero and 800 mm eccentricities as illustrated in Fig. 9.1(b). The initial prestressing steel stress is $0.6f_{pu}$, resulting in a total initial prestressing force of $P = 892$ kN. The axial force due to wall self weight and factored dead and live loads from the floors and roof is 578 kN. A grouted concrete masonry crushing strength of $f'_{m} = 18$ MPa is assumed, based on typical values encountered in New Zealand. Confining of the compression toe regions is achieved by embedding steel confining plates in the horizontal mortar joints, refer to Chapter 3. An initial elastic modulus of $E_m = 800f'_{m} = 14400$ MPa is assumed.

In order to investigate taller structures with longer fundamental period, an 8-storey PCM wall with the following dimensions was also investigated. The wall is 5.0 m long, 24 m high (3 m storey height) and 240 mm thick, and post-tensioned with eight 15.2 mm high strength prestressing strands located in pairs at -1000 mm, -400 mm, 400 mm and 1000 mm eccentricities. The initial prestressing steel stress is $0.68f_{pu}$, resulting in a total initial prestressing force of $P = 1360$ kN. The axial force due to wall self-weight and factored dead and live loads from the floors and roof is 1170 kN. Material properties of the prestressing steel and concrete masonry were similar to those of the 5-storey wall.
9.1.2 Finite element modelling

The prototype walls were modelled in DRAIN-2DX [9-1] using fibre elements, elastic beam elements, and inelastic truss elements. Extensive evaluation of the DRAIN-2DX fibre element (element 15) for modelling of unbonded post-tensioned concrete has previously been carried out successfully [9-2]. Because of the similarity between concrete and grouted concrete masonry material behaviour, such evaluation for PCM was deemed unnecessary.

The elements were defined according to the wall dimensions and materials. The vertical distribution of the fibre and beam elements is shown schematically in Fig. 9.1(b) for the 5-storey wall. Fibre elements were used at all construction joints, i.e. at the foundation level and at each intermediate floor levels, allowing for crack opening at these discrete locations. Beam elements were used elsewhere. Confined masonry material properties were used for the fibre element at the base (specially confined region) and unconfined masonry properties were used for all other fibre end beam elements (refer to Chapter 3). Fig. 9.2(a) shows the constitutive relationships assumed for the masonry.

Compression unloading properties for the individual fibres in the DRAIN-2DX fibre element are outlined in Fig. 9.2(b). In the elastic range the fibre unloads along the loading path, i.e. for any strain excursion before the fibre strain has once has surpassed the strain ε_m corresponding to the fibre maximum strength, f_m. For fibre strain excursions beyond ε_m, the unloading path is controlled by the stiffness degradation parameter FU (range of 0 to 1). Using FU = 0 results in unloading along a line pointing directly towards the origin (o). Using FU = 1 results in unloading along a line with a slope corresponding to the initial stiffness (defined by the line (o)-(i) in Fig. 9.2(b)). Using FU in the range between 0 and 1 results in unloading along a path with a slope based on linear interpolation between the slopes for FU = 0 and FU = 1. The lower bound of stress is given by the basis line which is defined by the origin (o) and the stress plateau (p) as shown in Fig. 9.2(b). Reloading occurs along the previous unloading path and is bound by the envelope. Kurame et. al. [9-2] recommended using FU = 0.5 for concrete based on research by Sinha et. al. [9-3]. In absence of relevant information on unloading properties specific to concrete masonry FU = 0.5 has been assumed to be applicable.

The lateral seismic mass of the structure, assumed lumped at each floor level, is given in Fig. 9.1(c). No attempt has been made to model the vertical and rotational seismic mass of the
structure. Modal analysis based on the wall initial stiffness EI resulted in a first mode natural period of $T_1 = 0.364$ sec and mode shape $\{\phi_1\}$ for the 5-storey wall, as shown in Fig. 9.1(d), and $T_1 = 0.621$ sec for the 8-storey wall.

9.1.3 Push-over analysis

Fig. 9.3 shows the result of a cyclic push-over analysis superimposed on the experimentally determined force-displacement response of Wall 1 of Series 3 (Chapter 7) using FEM fibre modelling. The figure reveals an experimental force-displacement response characterised with a distinct softening that occurs upon opening of a single crack at the base. It is noted that the
Fig. 9.3—Cyclic push-over response

Fig. 9.4—Force-displacement relationships

Wall returned to its original position and the initial stiffness was reinstated upon unloading, and that little hysteretic damping occurred. The FEM fibre model response shown in Fig. 9.3 modelled the experimental curve reasonably accurately in terms of initial stiffness and softening and the hysteretic shape was captured well for displacement up to approximately 0.040 m. Monotonic push-over analysis was performed on the multi degree of freedom (MDOF) structure (fibre model) shown in Fig. 9.1(b) by applying a lateral force vector with components at each floor level proportional to \( \{ \phi_1 \} \). Refer to Fig. 9.4 for the resulting envelope curve.
9.2 EQUIVALENT SINGLE-DEGREE OF FREEDOM STRUCTURE

It is desirable to transform the MDOF fibre model into a single degree of freedom (SDOF) structure. This allows for reduction of analysis effort and development of a simplified design approach. In the transformation to a SDOF structure, it is important to retain key structural properties observed in the MDOF model: base shear, base moment and structural period, so that the maximum displacement response $d_{\text{max}}$ at some equivalent height $h^*$ remains comparable between the MDOF and SDOF models.

9.2.1 Transformation

It can be shown that the $n^{th}$ mode response of a linear elastic MDOF structure can be modelled exactly by an equivalent SDOF structure with a modal mass of $M_n^*$, an equivalent height of $h_n^*$ and a uniform elastic stiffness of $E I$. Expressions for these quantities are [9-4]:

\[ M_n^* = \frac{(L_n^h)^2}{M_n} \quad \text{where} \quad L_n^h = \sum_{j=1}^{N} m_j \phi_{jn} \quad \text{and} \quad M_n = \sum_{j=1}^{N} m_j (\phi_{jn})^2 \]  
\[ h_n^* = \frac{L_n^\theta}{L_n^h} \quad \text{where} \quad L_n^\theta = \sum_{j=1}^{N} h_j m_j \phi_{jn} \]  
\[ \{m\} = \begin{bmatrix} m_1 \\ m_2 \\ m_3 \\ m_4 \\ m_5 \end{bmatrix} = \begin{bmatrix} 18.2 \\ 18.2 \\ 18.2 \\ 18.2 \\ 18.2 \end{bmatrix} \]  
\[ \{\phi_1\} = \begin{bmatrix} \phi_{15} \\ \phi_{14} \\ \phi_{13} \\ \phi_{12} \end{bmatrix} = \begin{bmatrix} 1.000 \\ 0.718 \\ 0.451 \\ 0.222 \\ 0.061 \end{bmatrix} \]
\[
\{h\} = \begin{bmatrix}
h_1 \\
h_2 \\
h_3 \\
h_4 \\
h_5 \\
\end{bmatrix} = \begin{bmatrix}
15 \\
12 \\
9 \\
6 \\
3 \\
\end{bmatrix} \text{ (m)}
\]  

(9.5)

In Eqns. 9.1 and 9.2, \(m_j, \phi_n\) and \(h_j\) correspond to the mass, modal displacement of mode \(n\) and the height above the base of floor level \(j\). The values of \(m, \phi_1\) and \(h\) are provided in Eqns. 9.3 to 9.5 for the MDOF structure shown in Fig. 9.1: From this the properties of an equivalent SDOF structure are (Fig. 9.1(e)):

\[
L_1^h = 44.68 \text{ t}, \quad L_1^\theta = 531.8 \text{ tm}, \quad M_1 = 32.28 \text{ t}, \quad M_1^* = 61.8 \text{ t}, \quad h_1^* = 11.90 \text{ m} \quad \text{and}
\]

\[
h_1^*/h_w = 0.793
\]

From experience [9-4], it was anticipated that higher mode effects (\(n = 2, 3, 4, \ldots\)) would not contribute significantly to the wall base moment. This implies that the maximum displacement of the wall could be modelled reasonably accurately using only the 1st mode SDOF structure for analysis, because the lateral displacement for cantilever wall ductile response arises nearly exclusively from flexural action adjacent to the base, thus \(h^* = h_1^*\) and \(M^* = M_1^*\).

Tables 9.1 and 9.2 show values of \(h^*/h_w, M^*/M_i\) and \(\phi_1\) calculated for walls with a range of number of stories \(N\), where \(h_w\) is the wall height and \(M_i\) is the total seismic mass. The assumptions were: all stories of identical height and constant flexural rigidity \(EI\) over the entire wall height. Results in Table 9.1 represent walls with seismic mass distributed uniformly at all floor levels and the roof while results in Table 9.2 represent walls with identical seismic mass \(m\) at all floor levels and \(m/2\) at the roof. It is observed that the results are independent of \(T, M_i, h_w\) and \(EI\).

### 9.2.2 Definition of SDOF structure

The next step in the process is to define the force-displacement relationship for the equivalent SDOF structure. It is clear that this relationship must be non-linear in order to capture the expected ductile behaviour. This conflicts with the assumption of using \(h^*\) to define the SDOF
structure because of the inherent linear elastic assumption. Nevertheless, the assumption also holds for ductile PCM wall response as will appear from the numerical analysis below.

The expected PCM wall behaviour represented by the MDOF fibre model is shown in Fig. 9.4 (envelope curve), along with two bi-linear curves. The curve denoted ‘ATC40’ was determined from the fibre element push-over curve according to the procedure outlined in the ATC40 document [9-5]. The procedure essentially equates the area under the push-over curve with the area under the bi-linear approximation, given a particular target point P on the push-over curve and that the steep branch of the bi-linear curve coincides with the push-over curve at 0.6$V_y$.

The curve denoted ‘Analytical’ originated from the simple analysis procedure proposed in Chapter 8, based on a wall initial stiffness of $K_1 = \frac{V_{cr}}{d_{cr}}$ a wall ‘post-yield’ stiffness of $K_2 = \frac{(V_y - V_{cr})}{(d_y - d_{cr})}$ and using $h_* = h^*$, refer to Fig. 9.5. It is seen in Fig. 9.4 that there is negligible difference between the two approximate curves. Because of the limited hysteretic damping produced by PCM walls, it is appropriate to use a bi-linear elastic model (loading and unload-
Fig. 9.5—5-storey wall, Analytical prediction model based on Chapter 8

Fig. 9.6—Bi-linear elastic model

9.3 DYNAMIC ANALYSIS

Time-history analyses were performed with the fibre MDOF and bi-linear elastic SDOF models defined above using NZS 4203:1992 [9-6] compliant earthquake records. Fig. 9.7 shows the NZS 4203 Uniform Hazard Spectra (elastic acceleration spectrum) for intermediate soils and a seismic zone factor of $Z = 1.2$ (zone of highest seismicity) with 3% and 5% viscous damping,
Fig. 9.7—Uniform Earthquake Hazard Spectra

specified for inelastic time-history analysis. The 5% damped spectrum is specifically given by NZS 4203 while the 3% spectrum was derived by scaling of the 5% spectrum as recommended by Kawashima and Aizawa [9-7]: $S_{ae}(T, \xi) = S_{ae}(T, \xi=0.05)[(1.5/(40\xi+1))+0.5]$. For reference, the UBC (1997) [9-8] elastic spectrum for seismic zone 4 and soil category $S_D$ with 5% viscous damping is also shown in Fig. 9.7, suggesting a similar level of seismicity. Both the NZS 4203 and the UBC spectra are associated with an earthquake return period of approximately 475 years.

As indicated, two viscous damping ratios $\xi$, 3% and 5%, were considered for analysis. The 5% spectrum shown in Fig. 9.7 represents the expected behaviour of nearly all ‘ordinary’ structures made of reinforced concrete, reinforced concrete masonry and structural steel. The origin of the ‘5%’ remains unclear. It has for example been suggested that 5% was selected in the ‘early days’ of electronic calculation because it resulted in numerical stability. Paulay and Priestley [9-9] stated that damping ratios for ductile reinforced concrete response generally range from 2% to 7%, such that 5% is readily justifiable. Kurama et al. [9-2] argued that 3% viscous damping was appropriate for dynamic analysis of unbonded post-tensioned concrete walls. Given the above statements it is recommended to adopt 3% viscous damping for unbonded PCM walls until further research confirms/rejects the appropriateness of using this viscous damping ratio.
It is clarified that $\xi$ in this study is completely dissociated from being a means of modelling of hysteretic damping. $\xi$ is attributed to energy dissipation throughout the structure due to effects such as concrete micro cracking, friction at molecular level (heat development) and deformation of non-structural elements, and occurs in the structure at any response level (elastic as well as inelastic). Hysteretic damping is, for reinforced concrete materials, mainly attributed to inelastic deformation of reinforcing steel, and solely is a feature of localised inelastic action in plastic hinges.

Viscous damping proportional to initial elastic stiffness was assigned to the models, with 3% mass proportional damping for the SDOF model and 3% viscous damping assigned to modes 1 and 2 for the MDOF model (Rayleigh damping). It is noted that no viscous damping was assigned to the base fibre element in the MDOF model because the expected large deformation response (rotation) would result in superficially high viscous damping in this element (critical for elements with significant stiffness degradation). Some amount of hysteretic energy can be dissipated in the fibre elements as suggested in Fig. 9.3 in case of inelastic fibre deformation. This effect provides only a small amount of energy dissipation in comparison with steel straining. The SDOF model did not have hysteretic energy dissipation ability.

9.3.1 Earthquake records

Four strong motion earthquake accelerograms were selected for analysis: El Centro 1940 NS, Taft 1952 S-69E, Hachinohe 1968 N-S and Matahina Dam 1987 N83E. Each accelerogram was normalised (frequency content modified while preserving phase angle) to comply with the NZS 4203 elastic 5% damped spectrum (using $Z = 1.0$) and termed ‘N1, N2, N3 and N4’. Plots of ground acceleration $a_g$ (fraction of g) vs. time of the normalised records ($Z = 1.2$) are shown in Fig. 9.8. It is seen that the maximum acceleration $a_{g,max}$ ranged between 0.43g and 0.51g, and that the duration of the strong ground motion records varied considerably. The strongest intensity (in terms of $a_g$) occurred at the beginning of the records for N1 and N2 while N3 showed relatively high intensity throughout the duration of the record. N4 is characterised by a short duration of strong intensity over approximately 8 seconds.

The 3% and 5% damped spectral responses of N1-N4 are shown in Fig. 9.9 with the hazard spectra. It is seen that there is good agreement between the response of the individual records and the hazard spectra, in particular for the 5% damped response. The average of the records...
Fig. 9.8—Normalised earthquake records N1:N4
Fig. 9.9—Spectral response of N1:N4, 3% and 5% damping

N1:N4 (shaded lines) show close correlation with both the 3% and 5% damped hazard spectra. The motivation for using these normalised (artificial) records was to reduce variation of response of the models as a result of variability in earthquake records and to ensure code compliance.

9.3.2 Time-History response: MDOF vs. SDOF

Fig. 9.10 shows the time-history response of the 5-storey MDOF and equivalent SDOF structures in terms of lateral displacement at h* due to N1 through N4. The SDOF bi-linear relation-
Fig. 9.10—5-storey, Displacement response MDOF vs. SDOF, N1:N4
ship was determined by the ‘Analytical’ procedure discussed above. Convergence of the time-history response was confirmed by analyses with varying time step length.

It is noted that the maximum response to N1 and N4 occurred at the same time for the MDOF and SDOF models and that the maximum displacements virtually were the same. The maximum SDOF response to N2 and N3 did not occur simultaneously with that of the MDOF, and the SDOF response was larger than the MDOF response.

Fig. 9.11 shows similar plots of time-history response for the 8-storey structures (3% damped). It is seen that the maximum response to N1:N4 occurred at the same time for the MDOF and SDOF models and that the maximum displacements were similar.

9.3.2.1 Displacement response

Tables 9.3 and 9.4 summarise results for both prototype structures and all four normalised earthquake records. It is seen that the maximum displacement response \( (d_{\text{max}}) \) varied significantly between the earthquake records, with the N1 record causing the largest displacement response for the 5-storey wall, the N4 record causing the largest displacement response for the 8-storey wall and the N2 record causing the least displacement demand for both walls.

---

**TABLE 9.3—Analysis results for 5-storey wall models, maximum values**

<table>
<thead>
<tr>
<th>EQ</th>
<th>MDOF Fibre model</th>
<th>SDOF Bi-linear elastic model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( d_{\text{max}}(h^*) )</td>
<td>( d_{\text{max}}(h^*) )</td>
</tr>
<tr>
<td>N1</td>
<td>0.148</td>
<td>0.113</td>
</tr>
<tr>
<td>N2</td>
<td>0.078</td>
<td>0.071</td>
</tr>
<tr>
<td>N3</td>
<td>0.079</td>
<td>0.101</td>
</tr>
<tr>
<td>N4</td>
<td>0.109</td>
<td>0.141</td>
</tr>
<tr>
<td>Average</td>
<td>0.103</td>
<td>0.133</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>0.028</td>
<td>0.036</td>
</tr>
<tr>
<td>StdDev/Av</td>
<td>0.28</td>
<td>0.27</td>
</tr>
</tbody>
</table>

**TABLE 9.4—Analysis results for 8-storey wall models, maximum values**

<table>
<thead>
<tr>
<th>EQ</th>
<th>MDOF Fibre model</th>
<th>SDOF Bi-linear elastic model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( d_{\text{max}}(h^*) )</td>
<td>( d_{\text{max}}(h^*) )</td>
</tr>
<tr>
<td>N1</td>
<td>0.226</td>
<td>0.299</td>
</tr>
<tr>
<td>N2</td>
<td>0.132</td>
<td>0.175</td>
</tr>
<tr>
<td>N3</td>
<td>0.233</td>
<td>0.308</td>
</tr>
<tr>
<td>N4</td>
<td>0.217</td>
<td>0.286</td>
</tr>
<tr>
<td>Average</td>
<td>0.202</td>
<td>0.267</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>0.041</td>
<td>0.054</td>
</tr>
<tr>
<td>StdDev/Av</td>
<td>0.20</td>
<td>0.20</td>
</tr>
</tbody>
</table>
Fig. 9.11—8-Storey, Displacement response MDOF vs. SDOF, N1:N4
paring the response of the MDOF and SDOF structures for a given earthquake, it is seen that reasonably consistent displacement demand arose between the SDOF and MDOF structures.

### 9.3.2.2 Drift demand

In the context of displacement focused design, it is important to evaluate the wall drift demand (also termed drift angle). Structural codes generally stipulate 'interstorey' drift limitations, $\gamma_{i,max}$, so that the structural response must, at any given time $t$ during the earthquake, conform to:

$$\gamma_{i,max} \geq \gamma_i = \max \left( \frac{d_j(t) - d_{j-1}(t)}{h_s} \right)$$  \hspace{1cm} (9.6)

where $d_j$ and $d_{j-1}$ symbolise the lateral displacements at floors $j$ and $j-1$, and $h_s$ is the height between these floors. Limitation of the interstorey drift demand $\gamma_i$ is intended to limit lateral deformation to avoid excessive floor slab and beam deformation, to avoid excessive damage to nonstructural elements such as windows, exterior wall cladding and interior partitions, and to limit P-Δ effects. The largest value for the right hand side (RHS) of Eqn. 9.6 is normally found near the top storey.

'Roof drift' is a measure of the global lateral deformation and is defined as follows:

$$\gamma_w = \frac{d_r}{h_w}$$  \hspace{1cm} (9.7)

In this equation, $d_r$ is the maximum lateral displacement at roof height and $h_w$ is the wall height. It is found that $\gamma_i$ always is larger than $\gamma_w$. For ductile cantilever walls of moderate storey height it is typically found that $\gamma_i$ is only slightly higher than $\gamma_w$ because most of the wall deformation results from development of a plastic hinge at the base.

It is interesting to calculate the drift demand at the equivalent height $h^*$ because it is available from both MDOF and SDOF analysis:

$$\gamma_{h^*} = \frac{d_{h^*}}{h^*}$$  \hspace{1cm} (9.8)

In this equation $d_{h^*}$ is the displacement demand at the height $h^*$.

Comparison of interstorey, roof and equivalent height drifts can be founded on the results shown in Tables 9.3 and 9.4. Comparing $\gamma_{h^*}$ and $\gamma_w$ for the MDOF structures it is revealed that
γ∗ is larger than γh∗ in all cases, as expected, and that the difference is minimal. Thus for ductile unbonded PCM walls, γh∗ is a good estimate for γ∗. Comparing γh∗ for the MDOF and SDOF structures it is seen that γh∗(SDOF) is a good approximation for γh∗(MDOF). Maximum interstorey drift was calculated for the fibre models and are shown in Tables 9.3 and 9.4. The results suggest that γ∗ on average is 8.2% larger than γh∗(MDOF). Given the above statements it is proposed that the interstorey drift demand on a ductile unbonded PCM wall can be approximately estimated by means of the wall drift at the equivalent height of the SDOF structure as follows:

\[
\gamma_i = 1.1 \times \gamma_{h^\ast}^{(SDOF)}
\]  

9.3.2.3 Base moment response

As expected, the maximum moment (M_{max}) demand did not vary significantly between the models when considering different earthquake records, simply because of the distinct base moment strength and low strain hardening (of the order of 1%). It appears that higher mode effects in the MDOF structure on average resulted in approximately 6-7% higher base moment, compared to that of the SDOF structure.

9.3.2.4 Base shear response

The base shear response was drastically different between the MDOF and SDOF models. Figs. 9.12 and 9.13 show the MDOF maximum storey shear (V_{MDOF}) normalised with SDOF maximum shear (V_{SDOF}) for the 5 and 8 storey structures and illustrate the large difference in shear demand between the MDOF and SDOF models. For the 5-storey wall, it was on average found that the MDOF maximum base shear (V_{base}) amounted to approximately 210% of that of the SDOF structure, and for the 8-storey wall the MDOF/SDOF maximum shear force ratio amounted to approximately 260%, clearly indicating a very strong influence of higher mode effects. It is also seen from Figs. 9.12 and 9.13 that V_{SDOF} was exceeded in all storeys, even in the top one.

Paulay and Priestley [9-9] explain that mode shapes in 2^{nd} and 3^{rd} modes of vibration of cantilevers with fixed or hinged bases are very similar, and that this suggests that the formation of a plastic hinge at the wall base may not significantly affect the response in the 2^{nd} and 3^{rd} modes. While a base plastic hinge will greatly reduce the wall actions associated with first mode response, it can be expected that those resulting from higher mode response of an inelastic cantilever will be comparable with elastic response actions. In reality, it is improbable that higher
Fig. 9.12—5-storey wall, storey shear

Fig. 9.13—8-storey wall, storey shear

Mode effects will contribute significantly to the moment distribution in the wall because of the short shear spans associated with the displacement patterns of higher modes and because of the distinct base moment strength (normally substantially lower than the strength required for strictly elastic response). In contrast, the 1st mode base shear is reduced by plastic hinging, while the higher mode shear contribution may still be significant. The storey shear amplification due to higher mode effects are undoubtedly dependent on the building height (or fundamental period T). Paulay and Priestley recommend a dynamic shear amplification factor, $\omega_n$, of 1.4 for 5-storey walls to 1.6 for 8-storey walls. More recent studies by Priestley and Amaris [9-10] on reinforced concrete cantilever walls suggest much higher dynamic shear amplification.
Values of $\omega_v = 1.8$ for 4-storey walls and $\omega_v = 2.5$ for 8-storey walls were found when comparing results from SDOF displacement based analysis with MDOF ductile time-history analysis. Research by Kurame [9-11] on precast unbonded concrete cantilever walls also confirm shear amplification of such magnitude.

The observations by Priestley and Amaris and by Kurame correspond well with the 5-storey and 8-storey analyses results given above. This holds despite the fact that unbonded PCM shows of significantly lower hysteretic damping and lower viscous damping (3% assumed) than RC and RCM, that in conjunction are likely to less effectively suppress higher mode effects. As will be shown in section 9.5, the achieved base moment reduction due to rocking (hinging adjacent to the base) relative to purely elastic response was 4 for the 5-storey wall and 5 for the 8-storey wall, thus the SDOF base shear was reduced 4-5 fold but the MDOF (real) base shear reduced with only 4/2.1 = 1.9 for the 5-storey wall and 5/2.6 = 1.9 for the 8-storey wall - clearly a significant contribution to the base shear from higher mode effects. It is finally noted that more accurate estimation of $\omega_v$ could be achieved by more accurate modelling of the actual wall shear stiffness, however this was beyond the scope of this study.

Given these results, it was concluded that the SDOF model, defined above, predicted the wall displacement response at $h^*$ and the base moment with reasonable accuracy, and that the base shear demand could suitably be predicted by estimating the amplification due to higher mode effects, e.g. using interpolation between $\omega_v = 2.1$ for the 5-storey wall ($T = 0.364$ sec) and $\omega_v = 2.6$ for the 8-storey wall ($T = 0.621$ sec) based on the building fundamental period $T$ (extrapolation may be used for buildings of period lower than $T = 0.364$ sec). Alternatively, the wall aspect ratio $A_r = h_w/l_w$ may simply be used for interpolation to find $\omega_v$ because this parameter is indicative of the building period and dynamic properties. It should be kept in mind that the proposed values are valid for force reductions $R$ of 4 to 5 (see next section for definition of $R$). Using $R = 1$ (purely elastic response) will result in $\omega_v$ slightly higher than 1.0 because the ‘full’ 1st mode actions (base moment and base shear) that dominate purely elastic response will be developed. Therefore, both the wall aspect ratio, $A_w$, and the achieved force reduction, $R$, should be considered when estimating $\omega_v$.

9.4 SPECTRAL ANALYSIS

Spectral analysis consists of calculating the nominal structural response as a function of the structural period $T$ for given structural characteristics. In the present case, the characteristics
were the strain hardening ratio $\alpha = K_2/K_1$ and the viscous damping ratio $\xi$. A parametric analysis has shown that the ratio $\alpha$ is typically of the order of 0.01 to 0.02. Calculations were performed for two viscous damping ratios: 3% and 5%.

The bi-linear elastic SDOF model, as illustrated in Fig. 9.6 was characterised by the following parameters: initial stiffness $K_1$, post-yield stiffness $K_2$, yield strength $V_y'$ and seismic mass $M^*$. It is noted that the yield strength signifies the wall strength at which wall response softens due to opening of the base crack; it is not associated with yielding of the prestressing steel.

### 9.4.1 Elastic response

The minimum wall yield strength $V_y$ required to ensure purely elastic wall response (a value strictly associated with developing the base moment $M_y = V_y h^*$ for the SDOF structure) was determined as the maximum wall strength demand for given $T$, $K_1$, $M^*$, $\xi$ and earthquake record (EQ). The elastic spectral acceleration $S_{ae}$ was defined as $V_y / g M^*$. It can easily be shown that $S_{ae} = \text{fn}(T, \xi, \text{EQ})$; noting that $S_{ae}$ is independent of choice of $K_1$.

The reader is reminded that the spectral acceleration for the elastic structure, $S_{ae}$, merely can be read off the Uniform Earthquake Hazard Spectra in Fig. 9.7 because of normalisation of the earthquake records.

### 9.4.2 Ductile response

The wall ductile response was calculated for a range of force reduction values $R$, defining the yield strength of the ductile SDOF structure as $V_y' = V_y / R$, where $V_y$ was determined from the elastic response as explained above. The definition of $V_y'$, $K_1$ and $K_2$ is shown in Fig. 9.6. A realistic range of $R$ values, 2 - 6, was selected, reflecting typically achieved force reduction for ductile reinforced concrete masonry cantilever walls. The spectral displacement $S_d = \text{fn}(T, R, \alpha, \xi, \text{EQ})$ was defined as the maximum recorded displacement for a given calculation and has the unit of length. It is observed that $S_d$ is independent of the choice of $K_1$ and $K_2$. Running multiple time-history SDOF analyses with various $K_1$ (but constant $M^*/K_1$ ratio) confirmed the independence of the choice of $K_1$ and $K_2$.

### 9.4.3 Spectral displacement

Fig. 9.14 shows the spectral displacement for $\alpha = 1\%$ and $\xi = 3\%$ and 5%. A period range of 0.1 sec to 1.0 sec was investigated, $R$ was varied between 2 and 6, and responses from the four
earthquake records were averaged. Defining the characteristic response value of $S_d$ as the average response is in line with the requirements for inelastic time-history analysis given in international documents and codes [9-5,9-6,9-8]. NZS 4203 specifically requires the use of a minimum of three earthquake records which must conform with the earthquake hazard spectrum (Fig. 9.7) in the period range of interest (a narrow period band around the structural fundamental period). UBC 1997 [9-8] requires that seven or more time-history analyses are performed before the average value of the response of the parameter of interest may be used for design. It was however felt for the present case, that averaging time-history response from the four normalised earthquake records was suitable because of expectation of more consistent response from such records than scaled natural records. As discussed above, four earthquake
records were used for analysis and each record conformed to the hazard spectrum for all periods investigated.

It is seen from Fig. 9.14 that the displacement demand increased with increasing structural period, and increased with increasing force reduction \( R \). Comparing the results for 3\% and 5\% damping, it is seen that \( S_d(3\%) \) generally was about 20\% higher than \( S_d(5\%) \). It is therefore essential to estimate the available damping, \( \xi \), realistically. Results of similar analyses using \( \alpha = 2\% \) and plotted in Fig. 9.15, showed only negligible reduction in \( S_d \), suggesting little influence of \( \alpha \) on the displacement response for \( \alpha \) in the range of 1\%-2\%. In the following only the case of \( \alpha = 0.01 \) is considered.
In order to better exploit the information presented in Fig. 9.14, the curves were fitted to power expressions that are plotted in Fig. 9.16. Note that this curve fitting is only valid in the 0.1 sec to 1.0 sec period range and should not be used for extrapolation beyond periods of 1.0 sec. These curves, referred to as design displacement spectra, represent a further abstraction of the analysis results. Nevertheless, these are regarded as realistic because there is no apparent reason for the curves not being smooth (similar to the hazard spectra given in Fig. 9.7).
9.4.4 Displacement ductility demand

The displacement ductility demand, $\mu_d$, on the idealised SDOF structure can be calculated by:

$$
\mu_d = \frac{d_{max}}{d_y} = \frac{S_d}{d_y/R} = \frac{S_dR(2\pi)^2}{S_{ae}gT^2}
$$

(9.10)

where $d_y = d_y/R$ is the yield displacement of the ductile SDOF structure, and $d_y = V_y/K_1$ the displacement corresponding to the required strength of the linear elastic SDOF structure. Fig. 9.17 shows $\mu_d$ (average) calculated for $R$ values of 1, 2, 4 and 6, $\alpha = 0.01$ and $\xi = 3\%$. It is seen from the figure that very large ductility demands arose in the short period range. For a period of 0.4 sec and $R = 4$, a displacement ductility demand of 10 was found for a ductile PCM structure. This can be compared to a typical value of $\mu_d = 4$ for a reinforced concrete masonry structure of similar period and force reduction. It is emphasised that unbonded PCM walls can, without difficulty, be designed to accommodate high ductility demand, because of their inherent behaviour characteristics: high displacement capacity and relatively high initial stiffness (see Chapters 5 - 7).
9.5 USE OF DISPLACEMENT SPECTRA

It is useful to regard a displacement, \( d_{tg} \), as the design target rather than a specified structural performance parameter in terms of \( R \) or \( \mu_d \) for reinforced masonry and concrete. Such an approach can be termed 'displacement focused design' and could proceed as follows:

Displacement focused design
1. Given wall gross dimensions and tributary lateral seismic mass, find \( T \),
2. Establish appropriate \( \xi \) (0.03 recommended),
3. Determine \( S_{ae} \) given \( \xi \) (Fig. 9.7 for example),
4. Calculate the elastic design base shear \( V_y = S_{ae}gM^* \) (SDOF only) (Tables 9.1 and 9.2),
5. Determine \( d_{tg} \) as the smaller of the estimated wall displacement capacity \( d_u \) (section 8.1.7) and code displacement (drift) limitation (e.g. [9-6,9-8]),
6. Enter the \( S_d \) chart (assume \( \alpha \)) at \( T, d_{tg} \) and find the appropriate \( R \) value (interpolation) (Fig. 9.16),
7. Design the wall for the base moment \( M_y' = (V_y/R)h* \),
8. Calculate and verify the assumed values of \( d_u \) and \( \alpha \). Some iteration may be necessary,
9. Design for (MDOF) base shear (section 9.3.2.4) to ensure sufficient shear strength and no sliding of the wall.

With regard to the structure shown in Fig. 9.1(c): \( T = 0.364 \) sec, \( \alpha = 0.01 \) and \( \xi = 3\% \), it was found from Fig. 9.7 that \( S_{ae} = 1.25 \), thus \( V_y = 759 \) kN. A wall drift limitation of 1.0% or \( d_{tg} = 0.112 \) m was assumed, for which Fig. 9.16 produced \( R = 4 \) (approximately), so that the design base moment would be \( M_y' = 2125 \) kNm and the peak design interstorey shear force is \( \omega \Delta V_y' = 2.1*759/4 = 398 \) kN. For comparison, the actual capacity was found to be \( M = 2240 \) kNm (or approximately \( V = 200 \) kN, see Fig. 9.4). The actual shear strength depends on the wall dimensions and shear reinforcement ratio and the base shear friction capacity depends on construction joint details. Thus the design objective of \( d_{tg} \) was achieved with the given design. \( \mu_d = 10.6 \) was calculated.

With regard to the 8-storey wall: \( T = 0.621 \) sec, \( \alpha = 0.01 \) and \( \xi = 3\% \), it was found from Fig. 9.7 that \( S_{ae} = 0.99 \), thus \( V_y = 1572 \) kN. A wall lateral displacement limitation of 0.2 m at \( h^* \) or \( d_{tg} = 0.200 \) m was assumed, for which Fig. 9.16 produced \( R = 5 \) (approximately), so that the design base moment would be \( M_y' = 5810 \) kNm and the peak design interstorey shear force is \( \omega \Delta V_y' = 2.6*1572/5 = 817 \) kN. For comparison, the actual capacity was found to be \( M = 5550 \)
kNm (or approximately $V = 300$ kN). Thus the design objective of $d_{og}$ was achieved with reasonable accuracy ($M$ 5% lower than $M_y'$) with the given design. $\mu_d = 10.5$ was calculated.

### 9.6 NOTES

Higher order effects must be taken into account in the wall design, in particular for determining the actual (MDOF) base shear demand, which may amount to 2 to 3 times $V_{\text{max}}$ from SDOF analysis. The maximum SDOF base shear $V_{\text{max}}$ and base moment $M_{\text{max}}$ are higher than $V_y'$ and $M_y'$, respectively, because of strain hardening. Strain rate dependency of the concrete masonry was not considered in this analysis. Detailed design of the entire wall should be conducted according to capacity design principles, e.g. [9-9].

### 9.7 CONCLUSIONS

It has been shown that a SDOF equivalent structure, based on a bi-linear elastic force-displacement relationship, can be used to accurately estimate the displacement response of multi-storey unbonded post-tensioned concrete masonry cantilever walls.

Displacement spectra were developed for PCM walls of realistic structural dimensions. It was found that the wall displacement demand was strongly dependent on the fundamental structural period ($T$), the force reduction factor ($R$) and the viscous damping ratio ($\xi$). Response was not sensitive to variation of the strain hardening ratio ($\alpha$) between 1% and 2%.

The displacement ductility demand on PCM walls was found to be much higher than that for reinforced concrete masonry walls of similar proportions.

With reference to the results from the detailed MDOF models, it was concluded that the wall base moment was determined with reasonable accuracy by the SDOF model, however, the base shear was underestimated by a factor of 2.1 to 2.6. The base shear amplification due to higher mode effects is subject to further investigation given the preliminary nature of this investigation.

It was concluded that the proposed analysis method is suitable for displacement focused design.
9.8 REFERENCES


Chapter 10

SUMMARY OF CONCLUSIONS

10. CONCLUSIONS AND FUTURE RESEARCH

The general scope set out in Chapter 1: "to provide detailed guidelines for complete analysis and design of ductile post-tensioned concrete masonry walls" was successfully achieved. Analysis and design of PCM walls can readily be conducted using the procedures proposed in Chapters 8 and 9. The analytical procedures were supported by experimental investigations, and international research and regulations.

This thesis investigated a broad array of PCM wall features, ranging from material properties and large scale structural testing to theoretical considerations. While the investigated elements were examined thoroughly, there are many more issues to be explored that fell outside the scope of this thesis. Some of these issues are listed in section 10.6.

Three series of wall tests confirmed that fully grouted unbonded post-tensioned concrete masonry is a competent construction form for ductile structural wall systems. The PCM walls reported herein exhibited a nearly non-linear elastic behaviour dominated by rocking response. Large drift capacity was measured: up to 1% was measured for U200 unconfined concrete masonry, more than 1% was measured for CP200 confined concrete masonry and 1.5% was measured for CP100 confined concrete masonry. Testing Series 3 that reflected a realistic 4-storey cantilever wall was particularly successful. All fully grouted wall tests showed that only localised damage in the lower wall corners occurred. Earthquake damage would therefore be easy to repair.

Partially and ungrouted PCM walls were capable of developing significant strength, exceeding the predicted nominal flexural strength. Significant displacement capacity was recorded despite their brittle failure mode.

Limited energy dissipation was observed during wall cycling. One wall from testing Series 2 incorporated energy dissipators that successfully increased the hysteretic energy dissipation.
Theoretical procedures for determining PCM wall in-plane force displacement-relationship successfully predicted the observed experimental behaviour. Inelastic time-history analysis using strong motion earthquake records revealed that PCM walls are likely to undergo significantly larger lateral displacements than reinforced concrete masonry walls of similar dimensions and strength. However, it was found, for a 5-storey PCM prototype building, that the expected drift demand was less than 1% and therefore lower than the typical code drift limitation of 1.5%. The 5-storey prototype model did not rely on any external energy dissipation.

Based on two creep and shrinkage testing series conducted at the University of Auckland and on international research, it was concluded that long term prestress losses are considerable in both grouted and ungrouted concrete masonry and must be taken into account in design. Use of high strength prestressing steel is required to minimise prestress loss.

Detailed conclusions for the individual chapters are presented below.

10.1 MATERIAL PROPERTIES - CHAPTER 3

Accurate prediction of $f_m'$ and $e_{m0}$ is necessary for design of PCM walls because of the significant influence of these parameters on the wall strength and displacement capacity. Unfortunately, the current New Zealand masonry code specifications poorly reflect actual concrete masonry properties (in particular strength and elastic modulus). For the time being it is recommended to base the concrete masonry properties on the rational methods presented in this document.

Theoretical stress-strain curves for unconfined and confined concrete masonry may be established using the Priestley-Elder formulation. The required information to do so is limited to $f_m'$ and, in case of confined masonry, the confining steel dimensions and yield strength. Using confinement plates in the bed joints greatly improves the masonry strain capacity. A uniaxial dependable strain capacity of more than 1% can readily be achieved.

10.2 TIME DEPENDENT EFFECTS - CHAPTER 4

For grouted concrete masonry it was recommended to use the creep coefficient $C_c = 3.0$, a value in agreement with the values stipulated by BS 5628 and AS 3700. For ungrouted concrete masonry $C_c = 3.0$ is similarly recommended for design purposes. Values predicted by BS 5628 and AS 3700 appear to be on the conservative side. The following shrinkage strains were
recommended for design under New Zealand conditions: for grouted walls $e_{sh} = 700 \mu \varepsilon$ and for un-grouted walls $e_{sh} = 400 \mu \varepsilon$, values in agreement with international research.

Creep and shrinkage properties for partially grouted concrete masonry should be based on interpolation between the results for grouted and un-grouted concrete masonry according to the ratio of grouted cells to the total number of cells.

Using high strength strand with an initial stress of $0.7f_{pu}$ and an axial load ratio of 0.15, about 20% loss of prestress can be expected after all long term losses have occurred. The use of high strength bar instead of strand, under similar conditions, is expected to result in about 30% pre-stress loss. Less loss is anticipated using ungrouted concrete masonry. In both grouted and ungrouted concrete masonry, losses are considerable and must be taken into account in design. However, high prestress loss can be countered by restressing of the prestressing steel at some later time after the initial stressing. It is, in theory, possible to reduce the prestress loss by about 50% by restressing about 3 months after the initial stressing.

10.3 STRUCTURAL TESTING - CHAPTERS 5, 6, AND 7

Series 1:

It was concluded that fully grouted unbonded post-tensioned concrete masonry is a competent material combination for ductile structural wall systems. The PCM walls reported herein exhibited a nearly non-linear elastic behaviour dominated by rocking response. Large drift capacity of more than 1.0% was measured. Only localised damage occurred.

Tendon force loss due to yielding should be expected for short (squat) walls at relatively low wall drift ratios. Even after tendon yielding, reliable and self-centring wall behaviour is expected. Before tendon yielding, little energy dissipation was observed during wall cycling.

Partially and ungrouted PCM walls were capable of developing significant strength, exceeding the predicted nominal flexural strength. Despite their brittle failure mode, significant displacement capacity was recorded.

Additional axial load from the prestressing enhanced the concrete masonry shear strength. The Paulay and Priestley [5-4] provisions for masonry shear strength was found to most accurately reflect experimental performance. The NZS 4230:1990 shear strength provision is highly conservative.
Series 2:

Strengthening of the flexural compression zones of fully grouted unbonded post-tensioned concrete masonry walls improved the wall displacement capacity and delayed the onset of strength degradation in comparison with PCM walls made with unconfined masonry. The PCM walls from Series 2 behaved similarly to the behaviour observed for testing Series 1; notably they exhibited a nearly non-linear elastic behaviour dominated by rocking response. The exception was P2-CP-CA-ED that exhibited rocking response with non-linear elasto-plastic behaviour due to the presence of energy dissipation devices.

The maximum wall strengths were insensitive to strengthening of the compression zone. Drift capacities of 0.7% to 2% were measured using CP200, 1.1% to 2.6% with CP100 and 1.1% to 1.4% with high strength block corners. The highest performing solution was CP100.

Only localised damage occurred. Relatively little energy dissipation was observed during cycling of all walls, except for P2-CP-CA-ED that had ‘dog-bone’ energy dissipation devices embedded. This simple device proved successful and provided a 33% wall strength increase and a threefold increase of hysteretic damping relative to P2-CP-CA.

Extreme masonry fibre strain at nominal flexural strength in the order of 0.0035 was found for CP200 and 0.0020 was found for CP100. Both values are substantially lower than 0.008 stipulated by NZS 4230:1990 for confined masonry. At ultimate displacement, extreme fibre strains as high as 0.046-0.050 were measured for both CP200 and CP100.

Series 3:

This testing series provided final confirmation that PCM walls of realistic proportions, strengthened in the flexural compression zones with confining plates (CP100), successfully can withstand severe cyclic loading as imposed by major earthquakes. Ductile response was measured, with reliable drift capacity of 1.5% for both walls tested. Relatively little energy dissipation was observed during cycling of the walls. Adequate shear friction between wall and foundation was achieved by intentional roughening of the wall to foundation interface.

All visual damage occurred to the lowest 300-400 mm in the flexural compression zone. Inelastic masonry response was measured in the lowest 400-600 mm of the walls. Extreme masonry fibre strain in the order of 0.0010 to 0.0020 were measured at nominal flexural strength. These values were substantially lower than 0.008 as stipulated by NZS 4230:1990 for...
confined masonry. Average strains in the extreme fibre as high as 0.019-0.024 were measured at ultimate displacement.

Comparison between PCM and RCM walls suggested that RCM walls are expected to exhibit larger displacement capacity than PCM walls primarily because of a longer plastic hinge length. PCM walls are expected to sustain much less and more localised damage during strong ground motion than RCM walls, resulting in simpler and cheaper post-earthquake repair.

10.4 ANALYSIS AND DESIGN - CHAPTER 8

For walls of low aspect ratio (say below 1.0), the predicted initial stiffness overestimates the actual wall stiffness. For aspect ratios above 2, accurate prediction is expected.

Accurate and consistent estimation of the force-displacement envelope (skeleton curve) can be achieved, using the predicted base shear and corresponding displacement at maximum serviceability moment, at nominal strength, at first tendon yield and at overstrength.

The predicted displacement capacity, $d_u$, was captured with reasonable accuracy and consistency (it is acknowledged that the prediction method in the first place was calibrated by the same tests). The strength $V_u$ associated with $d_u$ for confined concrete masonry (CP200/CP100) was captured reasonably accurately. The unconfined concrete masonry (U200) wall capacity at $d_u$ should be taken as 0.8$V_u$.

10.5 DYNAMIC ANALYSIS - CHAPTER 9:

A SDOF equivalent structure can be used to accurately estimate the displacement response of multi-storey unbonded post-tensioned concrete masonry cantilever walls. From an array of analyses, it was found that the wall displacement demand was strongly dependent on the fundamental structural period ($T$), the force reduction factor ($R$) and the viscous damping ratio ($\xi$). Response was not sensitive to variation of the strain hardening ratio ($\alpha$) between 1% and 2%.

The displacement ductility demand on PCM walls was found to be much higher than that for reinforced concrete masonry walls of similar proportions. Likewise, the base shear amplification factor of 2.1 to 2.6 recommended for PCM walls to account for higher mode effects, was much higher than that recommended for reinforced concrete masonry.
Displacement limitation is the primary focus of the proposed method. This makes the method suitable for displacement focused design where emphasis is put on displacement limitation rather than strength.

10.6 FUTURE RESEARCH

Experimental research:
Prediction methods described in Chapter 8 correlated well with the experimental results presented in Chapters 5, 6 and 7. Unfortunately, no experimental results were available to confirm that the predicted dynamic response, as presented in Chapter 9, reflects reality. It is therefore recommended to conduct dynamic shake table (scale model) testing. The wall displacement demand and base shear amplification are of particular interest in such investigation.

It is recommended to conduct further creep and shrinkage experiments. The current database was established under laboratory conditions and does not directly reflect the variability of the New Zealand exterior conditions. The influence of wall weather proofing on creep and shrinkage should also be addressed.

Only one wall had energy dissipators incorporated. While the wall test was successful, there remains further need to investigate this issue with regards to energy dissipator type, location and strength.

Theoretical work:
The displacement spectra developed in Chapter 9 covered a limited number of earthquakes and were only specific to medium soil conditions and a seismic zone factor of $Z = 1.2$. To expand the understanding and statistical reliability of the predicted PCM wall dynamic behaviour, a larger array of earthquake records, earthquake intensities, and soil types should be investigated. Displacement spectra reflecting additional external damping should also be developed systematically.

Energy dissipation arising from deformation of building floors should be investigated in detail. Damping from that source is normally disregarded for design of reinforced concrete masonry because of it’s low magnitude in comparison with the hysteretic damping occurring in the plastic hinge zone. For PCM walls with unbonded tendons that exhibit little hysteretic damping, the energy dissipation from yielding of floor slabs may have significant impact on the seismic displacement demand.
With reference to Chapter 9 results from the detailed MDOF models, it was concluded that the wall base moment was determined with reasonable accuracy by the SDOF model, however, the base shear was underestimate by a factor of 2.1 to 2.6. The base shear amplification due to higher mode effects should be explored further given the preliminary nature of this investigation.

The theoretical concepts developed in this thesis were specific to concrete masonry. Given the simplicity of the proposed analysis and design method, it would be obvious to port the method to analysis and design of unbonded post-tensioned concrete walls.
Appendix A

A. DEFORMATION AT NOMINAL FLEXURAL STRENGTH

The structural deformation at nominal flexural strength, $M_n$, notably lateral displacement of the top of the wall and vertical extension and shortening of the wall ends, can be evaluated by integration of wall curvature as illustrated by Fig. A.1. The moment varies linearly along the wall height when the lateral load, $V_F$, is assumed applied at the top of the wall. For a given axial load $(P+N)$ or axial load ratio, $\xi$, the moment-curvature relationship for wall flexure can be established, assuming that plane sections remain plane. The nominal strength limit state is defined by the maximum usable strain in the extreme masonry fibre at the wall base, in this study assumed to be 0.003 for unconfined concrete masonry and 0.008 for confined concrete masonry (refer to Chapter 3). Numerical integration of the curvature, as described in the following, allows for calculation of the wall lateral and vertical displacements at the top of the wall. Evaluation of the vertical deformation enables evaluation of the tendon force increase, $\Delta P$, due to structural deformation at nominal flexural strength. It is possible to present the result in a non-dimensional fashion so that any wall shape can be analysed.

It is acknowledged that a linear strain distribution inherently is associated with distributed flexural cracking, as suggested in Fig. A.2, which is consistently contradicted by testing showing

---

Fig. A.1—Definition of cantilever wall dimensions, forces and deformations
that only a single crack opens up at the base of the wall. Therefore one may regard the base crack as a ‘summation’ of the distributed cracking.

Method:
The approach consists of the following steps:

1. Definition of the stress-strain relationship for concrete masonry (\(\sigma-\varepsilon\))

2. Selection of axial force ratio \(\xi = \frac{P + N}{f_m I w h_w}\)

3. Calculation of moment curvature relationship (\(M-\phi\))

4. Numerical integration of curvature over the height of the wall to determine the lateral displacement, \(d_v\), and the vertical wall end extension and shortening, \(u_v\) and \(u_s\) (refer to definition in Fig. A.3)

Steps 2 through 4 are repeated for a realistic range of axial load ratios, \(\xi\): 0.05-0.25. Using the above procedure for both confined and unconfined concrete masonry (different \(\sigma-\varepsilon\) relationships), a series of normalised approximate equations can be generated.

Stress-strain relationship:
The stress-strain relationships used for the derivation is shown in Fig. 3.7 for both unconfined and confined concrete masonry, based on the Priestley-Elder relationship. It was assumed that \(\varepsilon_{pm}\) is limited to 0.004 for unconfined masonry. The confinement plate properties were \(p_s = 0.00577\) and \(f_{sh} = 240\) MPa. Further detail may be found in Chapter 3.
The moment-curvature relationship, \( M-\phi \), was calculated assuming that plane sections remain plane and that the masonry had no tensile strength. Fig. A.4 shows the moment-curvature relationships for two axial load ratios, assuming confined concrete masonry and the shown dimensions. The 'Original' curves, originating from moment curvature analysis, were modified slightly for numerical integration purposes and are denoted 'Modified'. The modification was based on a limiting moment taken as the average of the maximum moment and the moment corresponding to the ultimate masonry strain. Similar curves were generated for a range of axial load ratios, for both confined and unconfined masonry. The maximum curvature was based on a useable strain of \( \varepsilon_{mu} = 0.003 \) for unconfined concrete masonry and 0.008 for confined concrete masonry.

Lateral displacement:

The lateral displacement was evaluated numerically, based on integration of curvature:

\[
d_n = \int_{0}^{h_e} \phi(x)(h_e - x)dx
\]

(A.1)

where the curvature at any given height, \( \phi(x) \), corresponds to the moment \( M(x) \) as shown in Fig. A.1.

---

**Fig. A.3—Wall deformation at nominal flexural strength**
Fig. A.4—Moment-curvature relationships

Vertical displacement:

The vertical displacement of each end of the wall measured at the top of the wall, as shown on Fig. A.3, can be evaluated by integration of strain along the wall ends:

\[
\begin{align*}
    u_e &= \int_0^{h_e} \varepsilon_e(x) \, dx \\
    u_s &= \int_0^{h_s} \varepsilon_s(x) \, dx
\end{align*}
\]  

(A.2)

The distribution of vertical strain along the wall ends, \( \varepsilon_e(x) \) and \( \varepsilon_s(x) \), is easily derived in conjunction with the moment-curvature analysis.

Normalisation:

It can be shown that the wall lateral displacement at nominal flexural strength can be normalised with

\[
\frac{f_m' h_e^2}{E_m l_w}
\]  

(A.3)

and the vertical extension/shortening of the wall ends with

\[
\frac{f_m' h_e}{E_m}
\]  

(A.4)

Only the influence of the masonry elastic modulus, \( E_m \), appears not to be entirely linear because the deformation of the wall section adjacent to the base for moment near \( M_n \) is rather
controlled by the extreme masonry strain, $e_{mu}$. However, if wall deformation predictions are based on $E_m$ in the vicinity of $1000f''_m$, little error should result.

**Results:**

Following are the results of the analysis, presented in graphical form in Figs. A.5 and A.6. The thick lines represent the calculated response and the thin lines represent 2nd order polynomial approximations. The polynomial approximations given in Eqns. 8.20, 8.21, 8.24 and 8.25 were based on polynomial curve fitting.
Fig. A.5—Wall deformation at $M_n$ for unconfined concrete masonry
Fig. A.6—Wall deformation at $M_n$ for confined concrete masonry
Appendix B

B. FORCE-DISPLACEMENT PREDICTION - CALCULATION EXAMPLE

This appendix presents the detailed calculations for the force-displacement prediction example discussed in section 8.1.8. Reference to the applied equations are given. Wall dimensions, material properties, and initial stress and force are given in Figs. 8.9 and 8.10. Force and moment is given in kN and kNm, distance in m, stress in MPa and steel area in mm².

First cracking limit state:

Eqn. 8.1: \[ M_{cr} = \frac{(567 + 831)3.6}{6} = 839\text{kNm} \]

Eqn. 8.2: \[ V_{cr} = \frac{839}{10} = 83.9\text{kN} \]

Eqn. 8.4: \[ d_{cr} = \frac{2}{3} \frac{10^2(567 + 831)}{14400 \cdot 3.6^2 \cdot 0.19} + \frac{2(1 + 0.2)(567 + 831)}{5} \frac{14400 \cdot 0.19}{14400} = 0.0029\text{m} \]

Maximum serviceability moment:

Eqn. 8.6: \[ M_e = 2.04\left(0.5 - 1.21 \frac{2.04}{18}\right)3.6^20.19 = 1820\text{kNm} \]

\[ V_e = \frac{1820}{10} = 182\text{kN} \]

Eqn. 8.13: \[ d_e = \left(0.3 - 0.029 \frac{2.04}{18}\right) \frac{18\cdot10^2}{14400 \cdot 3.6} + \frac{12(1 + 0.2)10}{5} \frac{14400 \cdot 3.6 \cdot 0.19}{182} = 0.0108\text{m} \]

Nominal strength:

First iteration using \( \xi_n = 0.114 \):

Eqn. 8.25: \( u_e = 0.0117\text{m} \) and \( u_s = -0.00384\text{m} \)

Eqn. 8.26: \( \Delta P_1 = 10.1\text{kN}, \Delta P_2 = 8.5\text{kN}, \Delta P_3 = 7.0\text{kN}, \Delta P_4 = 5.5\text{kN}, \Delta P_5 = 3.9\text{kN} \)

and \( \Delta P = 35.0\text{kN}, e_t = 0.004\text{m} \Rightarrow \xi_n = 0.116 \)

Second iteration using \( \xi_n = 0.116 \):

Eqn. 8.25: \( u_e = 0.0115\text{m} \) and \( u_s = -0.00387\text{m} \)
Eqn. 8.26: $\Delta P_1 = 9.8 \text{kN}, \Delta P_2 = 8.3 \text{kN}, \Delta P_3 = 6.8 \text{kN}, \Delta P_4 = 5.3 \text{kN}, \Delta P_5 = 3.8 \text{kN}$

and $\Delta P = 34.0 \text{kN}, c_t = 0.004 \text{m} \rightarrow \xi_n = 0.116 \text{ (OK)}$

Eqn. 8.18: $a = \frac{831 + 34 + 567}{0.972 \cdot 18 \cdot 0.19} = 0.431m$

Eqn. 8.17: $M_n = (831 + 34) \left(\frac{3.6}{2} + 0.004 - \frac{0.431}{2}\right) + 567 \left(\frac{3.6}{2} - \frac{0.431}{2}\right) = 2272 \text{kNm}$

$V_f = \frac{2272}{10} = 227 \text{kN}$

Eqn. 8.19:

$d_n = (7.63 \cdot 0.116^2 - 5.40 \cdot 0.116 + 1.69) \cdot \frac{12 \cdot (1 + 0.2)10}{14400 \cdot 3.6} - \frac{18 \cdot 10^2}{14400 \cdot 3.6} = 0.0412m$

Stress in tendon furthest away from compression end of wall:

$f_{ps1} = \frac{(P_1 + \Delta P_1)}{A_{ps1}} = \frac{831/5 + 9.8}{140} = 1257 \text{MPa}$

Stress in tendon closest to compression end of wall:

$f_{ps5} = \frac{(P_5 + \Delta P_5)}{A_{ps5}} = \frac{831/5 + 3.8}{140} = 1214 \text{MPa}$

First tendon yield:

$c = a/\beta = 0.431/0.96 = 0.449 \text{ m}$

Eqn. 8.31: $d_{ty} = \frac{1517 - 1257}{190000 \cdot 10/15} \cdot \frac{10^2}{3.6/2 + 0.4 - 0.449} = 0.1172 \text{m}$

where $h_v/h_w = 10/15$ modifies $E_{ps}$ to reflect the actual tendon length.

Eqn. 8.32: $\Delta P_{ty1} = (1517 - 1257)140 \cdot \frac{3.6/2 + 0.4 - 0.449}{3.6/2 + 0.4 - 0.449} = 36.4 \text{kN}$

$\Delta P_{ty2} = 32.2 \text{kN}$

$\Delta P_{ty3} = 28.1 \text{kN}$

$\Delta P_{ty4} = 23.9 \text{kN}$

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\[ \Delta P_{ty5} = 19.8kN \]

Eqn. 8.33: \[ \Delta P_y = 140.4kN \]

Eqn. 8.35: \[ a_y = \frac{831 + 140 + 567}{0.972 \cdot 18 \cdot 0.19} = 0.463m \]

Eqn. 8.34: \[ M_{ty} = 36.4(3.6/2 + 0.4) = 19.8(3.6/2 - 0.4) - \frac{0.463}{2} \cdot 140 = 229kNm \]

Eqn. 8.36: \[ M_y = (831 + 34 + 567)\left(\frac{3.6}{2} - \frac{0.463}{2}\right) + 229 = 2475kNm \]

\[ V_y = \frac{2475}{10} = 248kN \]

Eqn. 8.37: \[ d_y = 0.041 + 0.1172 = 0.158m \]

Overstrength:

Eqn. 8.39: \[ P_y = 5 \cdot 140 \cdot 1517 = 1062kN \]

\[ a_o = \frac{1062 + 567}{0.972 \cdot 18 \cdot 0.19} = 0.490m \]

Eqn. 8.38: \[ M_o = (1062 + 567)\left(\frac{3.6}{2} - \frac{0.490}{2}\right) = 2533kNm \]

\[ V_o = \frac{2533}{10} = 253kN \]

Eqn. 8.40: \[ d_o = \frac{1517 - 1214}{190000 \cdot 10/15} \cdot \frac{10^2}{3.6/2 - 0.4 - 0.490} = 0.310m \]

Ultimate displacement capacity:

Assume: \[ c = \frac{\frac{1}{2}(a_y + a_o)}{\beta} = \frac{\frac{1}{2}(0.463 + 0.490)}{0.96} = 0.496m \]

The variation of \( c \) between the first tendon yield and overstrength limit states is minimal because of little axial force variation.

\[ d_u = \frac{0.76\left(10 - \frac{0.76}{2}\right)}{0.496 - 0.013} = 0.192m \]
The wall strength at $d_u$ is found by interpolation between first tendon yield and overstrength limit states with respect to displacement:

$$V_u = V_y + \frac{V_o - V_y}{d_o - d_y} (d_u - d_y) = 248 + \frac{253 - 248}{0.310 - 0.158} (0.192 - 0.158) = 249\,kN$$
Appendix C

C. COMPARISON OF PREDICTED VS. EXPERIMENTAL BEHAVIOUR

C.1 FLEXURE/ROCKING PREDICTION METHOD

Only fully grouted walls that failed in flexure are incorporated in the present comparison.

C.1.1 Assumptions

Following assumptions were made: minimum unconfined masonry strength, $f'_m$, of 16 MPa, plastic zone height, $h_p$, of 200 mm for the single storey walls and 400 mm for the three storey walls, ultimate masonry compression strain, $\varepsilon_{mu}$, of 0.020 for U200 and CP200 and $\varepsilon_{mu}$ of 0.013 for CP100 masonry. All other wall details may be found in Chapters 5, 6 and 7.

Predictions of the wall force-displacement characteristics were carried out according to the formulae presented Chapter 8. The only deviating assumption was made for FG-L3.0-W15-P2-CPC that had constant force applied to the prestressing bars, consequently $M_n$ was based on N+P and the base shear and displacement at first tendon yield and overstrength were not calculated. All walls in Series 2 and 3 were confined with CP200 (K = 1.04) or CP100 (K = 1.08), except for FG-L3.0-P2C-HB from Series 2 which had one high strength blocks incorporated in each wall corner and was calculated using CP200 confined concrete masonry properties.

C.1.2 Comparison

Each figure in the following shows comparisons of the global response and response for loading up to nominal strength. In these curves, the thick line connecting the markers describes the predicted response (envelope/back bone curve) with the markers representing the following limit states: square = overstrength, triangle = first tendon yield, diamond = nominal strength, solid circle = maximum serviceability, hexagon = first cracking and circle = ultimate displacement capacity. The experimentally determined force displacement curves are shown in thin line.
C.1.2.1 Series 1 walls

Figs. C.1-C.5 show comparison of predictions vs. recorded force-displacement behaviour for Series 1 testing of unconfined single storey walls.

In terms of global measured response vs. prediction, it is clear from Figs. C.1-C.5 that there is good correlation between experiments and predictions at the maximum serviceability, nominal strength and tendon yield limit states.

In terms of wall response beyond tendon yield, it is seen that the predicted strength at the predicted wall displacement capacity (circle located on the prediction curve) was only achieved for one wall, notably FG-L3.0-W20-P3, which had a relatively low axial load applied to it. A lower bound strength at wall displacement capacity, $d_u$, was estimated using $\alpha = 0.5$ and $\beta = 1.0$ in for calculation of the wall flexural compression zone length $c_u$ (centroid of compressive stress located at a distance of $\frac{2}{3}c_u$ away from the extreme compression fibre). These results are shown with the circular markers slightly offset vertically from the circles on the prediction curves.

In terms of the predicted wall displacement capacity, it is seen from Figs. C.1-C.5 that initiation of strength degradation generally is captures well. Only wall FG-L3.0-W20-P3 showed significantly large displacement capacity than predicted. This wall differed from the other walls in series 1 in the sense that the maximum wall axial force due to two yielding tendons (third tendon located in the wall compression zone and had nearly lost all prestress at that stage) was applied to a larger wall section, thus resulting in a shorter compression zone and a proportionally lower extreme masonry fibre strain.

It is seen from the low level plots in Figs. C.1-C.5 that the predicted wall initial stiffness represented by the ‘first cracking’ limit state (hexagonal marker) in all cases exceeded the measured initial stiffness. This discrepancy is mainly attributed to inadequacy of the plane section assumption to describe deformation of walls of low aspect ratio. This observation is confirmed by studying the low level response of the wall with the highest aspect ratio, FG-1.8-W15-P2, in Fig. C.5 which indicates an accurate prediction of the measured initial stiffness.
Fig. C.1—Series 1 FG-L3.0-W20-P3: Prediction vs. experimental behaviour
Global response

Low level response

Fig. C.2—Series 1 FG-L3.0-W15-P3: Prediction vs. experimental behaviour
Fig. C.3—Series 1 FG-L3.0-W15-P2C: Prediction vs. experimental behaviour
Drift (%)

Global response

Low level response

Fig. C.4—Series 1 FG-L3.0-W15-P2E: Prediction vs. experimental behaviour
Fig. C.5—Series 1 FG-L1.8-W15-P2: Prediction vs. experimental behaviour
C.1.2.2 Series 2 walls

Figs. C.6-C.8 show experimental results and predictions for three (four) single storey walls with strengthened corners.

With regards to global measured response vs. prediction, it appears from Figs. C.6-C.8 that there is good correlation between experiments and predictions at the maximum serviceability, nominal strength, overstrength and tendon yield limit states.

In terms of wall response beyond the overstrength limit state, it is seen that the predicted strength at the predicted wall displacement capacity, $d_u$ (circle located on the prediction curve) was achieved for all walls in this test series. The lower bound strength at wall displacement capacity, $d_u$, estimated as $0.8V_u$, appears to significantly underestimate the wall strength.

In terms of the predicted wall displacement capacity, it is seen from Figs. C.6-C.8 that initiation of strength degradation generally is captures reasonably consistently.

It is seen from the low level plots in Figs. C.6-C.8 that the predicted wall initial stiffness represented by the 'first cracking' limit state (hexagonal marker), also for the confined walls, in all cases exceeded the measured initial stiffness. As was explained above, this discrepancy is attributed to inadequacy of the plane section assumption to describe the deformation of walls of low aspect ratio.

C.1.2.3 Series 3 walls

Figs. C.9 and C.10 show experimental results and predictions for the two 3-storey walls with strengthened corners.

With regards to global measured response vs. prediction, it appears from Figs. C.9 and C.10 that there is good correlation between experiments and predictions at the maximum serviceability, nominal strength and tendon yield limit states. It is observed in these figures that the measured strengths were considerably higher than the predicted strength. This was attributed to friction in the testing setup. The overstrength limit state was not reached in any of the Series 3 wall tests.
Fig. C.6—Series 2 FG-L3.0-W15-P2-CP: Prediction vs. experimental behaviour
Fig. C.7—Series 2 FG-L3.0-W15-P2-CP-CA: Prediction vs. experimental behav.
Fig. C.8—Series 2 FG-L3.0-W15-P2-HB: Prediction vs. experimental behaviour
In terms of wall response beyond the overstrength limit state, it is seen that the predicted strength at the predicted wall displacement capacity, $d_u$, (circle located on the prediction curve) was achieved for both walls in this test series. This was expected because the prediction model was calibrated with these two wall tests. The lower bound strength at wall displacement capacity, $d_u$, estimated as $0.8V_u$, appears to significantly underestimate the wall strength. The initiation of strength degradation generally was captures well.

It is seen from the low level plots in Figs. C.9 and C.10 that the predicted wall initial stiffness, represented by the ‘first cracking’ limit state (hexagonal marker), in both cases captured the actual response.

**C.1.3 Conclusion**

The following conclusions were drawn from comparison between predictions using the procedures developed in this chapter and experimental results.

**Initial stiffness:**

For wall of low aspect ratio (say below 1.0), the predicted initial stiffness based on $V_{cr}$ and $d_{cr}$ overestimates the actual wall stiffness by a factor of the order of 1.5. For aspect ratios above 2, accurate prediction is expected.

**Global response:**

Using the predicted base shear and displacement at maximum serviceability moment, nominal strength, first tendon yield and overstrength, an accurate estimation of the force-displacement envelope (skeleton curve) can be achieved.

**Displacement capacity:**

The predicted displacement capacity, $d_u$, was captured with reasonable accuracy using the values for extreme fibre strain and plastic zone length given in Table 8.1. The strength associated with $d_u$ for CP100 confined concrete masonry can be based on $V_u$. For CP200 confined masonry and U200 unconfined masonry, it appears prudent to limit the wall strength (base shear) at $d_u$ to $0.8V_u$. The maximum strength predicted at $d_u$ should be limited by $M_o$. 
Fig. C.9—Series 3 S3-1: Prediction vs. experimental behaviour
Fig. C.10—Series 3 S3-2: Prediction vs. experimental behaviour