THE SEISMIC DESIGN OF PLYWOOD SHEATHED SHEAR WALLS
• p.231, Figure 5.5:
  (a) Load versus Total deflection. (b) Lead versus
  lozenging deflection should read (a) Load versus lozenging
deflection. (b) Load versus Total deflection.

• p.306, last line:
  Eq. 5.1, should read Eq. 5.7.

• p.308, Second line from bottom:
  subtracting should read subtracting

• p.319, Figure 5.45 The hold-down force:
  \( \frac{V_{0U}}{B} \) should read \( \frac{V_{0H}}{B} \)
• p.48: Replace Fig. 2.17 by:

• p.57, Equation 2.69: \[ \sum \text{nails} \] should read \[ \sum \text{BC} \]

• p.68, Second paragraph, Third line from the bottom: \[ k_e \] should read \[ K_e \]

• p.108, Figure 3.21: Title should read Cyclic behaviour...

• p.143, Third line: \[ f'_b,1', f'_b,2 \] should read \[ f'_B,1', f'_B,2 \]

• p.164, First Paragraph, second line from the bottom: diameter of sheathing should read diameter to sheathing.

• p.193, Table 4.5 fourth row: 150 x 150 should read 150 x 50
ERRATA
March 1987

• p. XXXIV, Reference 34:
  In-plane cyclic should read In-plane cyclic.

• p. 2, Second paragraph, first line:
  [7, 10] should read [8, 10].

• p. 23, Fifth line:
  $k_e$ should read $K_e$

• p. 24, Equation 2.11:
  $\Delta_x$ should be $\Delta_x$

• p. 27, Equation 2.18:
  $\sin \left( \frac{1}{H} y \right)$ should read $\sin \left( \frac{i \pi y}{H} \right)$

• p. 28, Four lines up from the bottom:
  $k_e$ should read $K_e$

• p. 28, Six lines up from the bottom:
  $k_e$ should be $K_e$

• p. 39, Third line:
  and shows the simplified should read and shows that the simplified.
THE SEISMIC DESIGN OF PLYWOOD

SHEATHED SHEAR WALLS

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

at the
University of Canterbury

by
WAYNE GAVIN STEWART

University of Canterbury,
Christchurch, New Zealand

1987
ABSTRACT

A design methodology for earthquake resistant plywood sheathed shear walls is presented. The sheathing nailing is selected as the ductile load limiting element, whereby the large displacement demands imposed during an earthquake can be sustained without failure of the timber members in bending, shear or tension. Analytical models for predicting the elastic behaviour and ultimate strength of shear walls are formulated in order to develop the design procedure.

An experimental study of nailed sheathed joints was undertaken, and showed that such joints were able to sustain large reverse cycles well into the inelastic range. Complementary to the experimental study, an analytical strength model for nailed joints is described, and identifies nail diameter, nail length and nail coating as being the variables most influential on joint strength.

Eleven full scale plywood sheathed shear walls were subjected to reverse cyclic quasi-static loading and shaketable excitation. The performance of the sheathing nailing, framing connections and anchorage connections is reported in detail. The test walls exhibited progressive stiffness degradation resulting in pinched hysteresis loops, prior to failure through the nail heads pulling through the plywood, or the nail point withdrawing from the framing.

A theoretical time-history single degree of freedom idealisation is described to predict the dynamic response of shear walls. Theoretical predictions compared well with the experimental shaketable behaviour of the full scale test walls.

The displacement demands on shear walls which exhibited pinched hysteresis loops are compared with the corresponding displacement demands on an elastoplastic structure.
"I thank thee, and praise thee, O thou God of my fathers, who hast given me wisdom and might, and hast made known unto me now what we desired of thee."

Daniel 2:23
ACKNOWLEDGEMENTS

The research described in this thesis was carried out in the Department of Civil Engineering, University of Canterbury, under the overall guidance of its Head, Professor R Park.

I wish to record my appreciation to Dr John Dean for his encouragement and guidance during the course of the project, and consider it a privilege to have been associated with him.

My thanks are also extended to Dr A Carr, Dr A Buchanan and Dr P Moss for their assistance and useful suggestions during the project, to Mr Dai Ruitong of the Shanghai College Branch of Tonggi University, China, for his interest in the project, and assistance in part of the experimental work, and to Mr Steve May who read the manuscript.

I wish to thank the Technical Staff of the Civil Engineering Department, especially Messrs G Hill, P Coursey and T Scott, and of the Forestry School, especially Dr J Walker and Mr P Fuller, for their assistance in the experimental programme. I also wish to thank Mr L Gardner for the photographic work, and Mrs Jan Stewart for typing the tables.

My thanks are extended to the Ministry of Works and Development Staff of the Christchurch District Office, especially the District Civil Engineer, Mr D Tucker, for his co-operation in the preparation of the manuscript; Mrs E Ransfield, Mrs D Hogan, Mrs Y Savage and Mrs K Rouch for typing the manuscript; and Mr R Meekin and Mrs F Sanders for the draughting work.

The generous financial assistance provided by the Ministry of Works and Development is gratefully acknowledged.

Finally, the project could never have been achieved without the encouragement and support of my wife, Julie.
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NOTATIONS

A = Effective area of framing member at the critical section.

A = Steady-state response amplitude.

A_g = Excitation or shaketable amplitude.

A_n = Contact area between embedded nail shank and timber.

A_s = Slackness amplitude.

B = Width of sheathing.

C = Sheathing nail spacing.

C = Viscous damping coefficient.

C_r = 2\omega / M_n = Critical damping coefficient.

C = Basic seismic coefficient.

C_b = V_e / Mg = Base shear coefficient.

d_n = Nail diameter.

D = A / A_g = Dynamic magnification factor.

E = Modulus of Elasticity for framing members.

f = Nail withdrawal resistance factor.

f_b = Bearing stress.

f_b = Timber bearing strength.

f_{b} = Maximum flexural stress in framing member, may be further subscripted E for edge framing stud, I for interior framing stud, or p for framing plate.
\( f'_b \) = Maximum flexural stress in framing member at the wall ultimate strength \( V' \), may be further subscripted \( E \) for edge framing stud, \( I \) for interior framing stud or \( p \) for framing plate.

\( f'_b \) = Timber bearing strength in the direction normal to the nail shank.

\( f'_o \) = Maximum framing stress at the wall overstrength \( V'_o \), may be further subscripted \( b \) for flexure, and \( t \) for tension.

\( f_{o,t(M,\text{max})} \) = Axial stress in framing stud at the position of maximum bending moment.

\( f'_s \) = Average shear stress in sheathing, or shear stress in framing members, may be further subscripted \( E \) for edge framing stud \( I \) for interior framing stud, or \( p \) for framing plate.

\( f'_s \) = Shear stress in framing members at the wall ultimate strength \( V' \), may be further subscripted \( E \) for edge framing stud, \( I \) for interior framing stud or \( p \) for framing plate.

\( f'_t \) = Maximum tensile stress in framing member, may be further subscripted \( E \) for edge framing stud, \( I \) for interior framing stud.

\( f'_t \) = Maximum tensile stress in framing members at the wall ultimate strength \( V' \), may be further subscripted \( E \) for edge framing stud or \( I \) for interior framing stud.

\( f_{ut} \) = Ultimate tensile strength of nail shank.

\( f_y \) = Yield stress of nail shank.

\( f_1', f_2 \) = Frequencies at the intercepts of the response curve with the horizontal line \( D_{\text{max}}/\sqrt{2} \).

\( F \) = Strength of non-ductile components within a shear wall.
\( F' \) = Basic permissible stress, may be further subscripted -
    b for framing flexural stress
    p for perpendicular-to-grain bearing stress
    s for sheathing shear stress

\( F_b \) = Friction force arising from normal bearing load \( N_b \).

\( F_d \) = Dependable strength, may be further subscripted -
    b for framing flexural strength
    p for perpendicular-to-grain bearing strength
    s for sheathing shear strength
    t for framing tension strength
    x for horizontal framing joint strength
    y for vertical framing joint strength

\( F_p \) = Belt friction.

\( F'_w \) = Basic Permissible withdrawal load for single nail.

\( g = 9.81(\text{m/s}^2) \) Acceleration of gravity.

\( G \) = Modulus of Rigidity for sheathing.

\( H \) = Height of sheathing.

\( I \) = Second moment of area for framing about the axis in the
    plane of the shear wall.

\( k \) = Stiffness of nail joint.

\( k_d \) = Degrading stiffness.

\( k_o \) = Elastic Stiffness of Wall.

\( k_p \) = Pinching stiffness.

\( k_s \) = Spring stiffness of slackness oscillator, or Secant
    stiffness of nailed joint.

\( K_b \) = Initial stiffness of bearing stress-embedment curve.
\[ K_d \quad \text{= Loading rate factor.} \]

\[ K_e \quad \text{= Elastic Stiffness Parameter, may be further subscripted E for edge framing stud or I for interior framing stud.} \]

\[ K_I \quad \text{= Inelastic stiffness parameter, may be further subscripted E for edge framing stud or I for interior framing stud.} \]

\[ K_t \quad \text{= Nail type factor.} \]

\[ K_w \quad \text{= Moisture content factor.} \]

\[ l_e \quad \text{= Effective penetration length of nail shank.} \]

\[ l_n \quad \text{= Nail length.} \]

\[ l_p \quad \text{= Nail point length.} \]

\[ l_z \quad \text{= Length of crushed timber within framing member.} \]

\[ m \quad \text{= Mass of steel base beam.} \]

\[ \text{max, min} \quad \text{Subscripts denoting maximum or minimum values of a variable.} \]

\[ M \quad \text{= Inertia mass.} \]

\[ M_{o,s} \quad \text{= Maximum bending moment in framing stud at the wall overstrength } V_o. \]

\[ M_p \quad \text{= Plastic moment of nail shank.} \]

\[ M_p' \quad \text{= Bending moment in framing plate member at wall ultimate strength } V'. \]

\[ M_s \quad \text{= Bending moment in framing stud member, may be further subscripted E for edge framing stud or I for interior framing stud.} \]
\( M_s' \) = Bending moment in framing stud member at Wall ultimate strength \( V' \), may be further subscripted \( E \) for edge framing stud or \( I \) for interior framing stud.

\( n \) = Number of sheathing panels.

\( N_{b} \) = Normal bearing load.

\( N_{p} \) = Normal force acting on nail at the plastic hinge.

\( N_{r} \) = Radial pressure acting on embedded nail shank.

\( P \) = Lateral load of nailed joint.

\( P' \) = Ultimate strength of sheathing nails, may be subscripted \( x \) for the component of force in \( x \) direction and \( y \) for component of force in \( y \) direction.

\( P_{av} \) = Average nail force parallel to panel edges.

\( P_{b} \) = Nail bearing load.

\( P_{B} \) = Basic Permissible shear load for single nail.

\( P_{E} = 1.25 \frac{P_{WS}}{P_{WS}} \) = Design earthquake nail load.

\( P_{H,WS} \) = Permissible Wind/Seismic Load for Hold-down connection.

\( P_{j,\text{max}} \) = Maximum nail force in shear wall.

\( P_{O} \) = Overstrength of nailed joint or Yield load parameter controlling nailed joint behaviour in Finite Element Analysis [24].

\( P_{P} \) = Axial force in framing plate member.

\( P_{P}' \) = Axial force in framing plate member at wall ultimate strength \( V' \).

\( P_{S} \) = Axial force in framing stud member, may be subscripted \( E \) for edge framing stud or \( I \) for interior framing stud.
\[ p_s' = \text{Axial force in framing stud member at wall ultimate strength } V' \text{ may be further subscripted E for edge framing stud or I for interior framing stud.} \]

\[ P_{WS} = \text{Permissible wind/seismic load for nail driven into side grain of timber.} \]

\[ P_y = \text{Yield load of nailed joint.} \]

\[ P_1 = \text{Bilinear stiffness parameter controlling nailed joint behaviour in Finite Element Analysis [24].} \]

\[ r_{c,x}, r_{c,y} = \text{Framing connection slip ratios in the x and y directions.} \]

\[ r_s = \Delta'_y / \Delta'_x = \text{Nail slip ratio, may be further subscripted s for framing stud member or p for framing plate member.} \]

\[ r_{un} = \text{Unloading factor.} \]

\[ r_u = \mu_{\Delta_P}/\mu_{\Delta_{EP}} = \text{Ductility ratio.} \]

\[ r_{u,m} = \mu_{\Delta_{Pm}}/\mu_{\Delta_P} = \text{Modified ductility ratio.} \]

\[ r_1 = \text{Bilinear factor.} \]

\[ r_2 = \text{Trilinear factor.} \]

\[ R = \text{Risk factor used in determining design base shear } V'_E, \text{ or Force reduction factor.} \]

\[ R_n = \text{Random number between 0 and 1.} \]

\[ SM = \text{Combined structural type and material term used in determining design base shear } V'_E. \]

\[ t_f = \text{Framing member thickness.} \]

\[ t_s = \text{Sheathing thickness.} \]

\[ T_n = \text{Nail Tension.} \]

\[ T_0 = \text{Initial natural period of vibration.} \]
$T_P$ = Nail tension due to belt friction.

$T_s$ = Spring stiffness natural period.

$T_w$ = Withdrawal resistance of nail shank.

$U$ = Strain energy, may be subscript -
  f framing stud member, or
  n sheathing nailing

$V$ = Horizontal wall load.

$V'$ = Horizontal wall ultimate strength.

$V_{acc} = M\ddot{Y}(t)$ = Wall base shear determined from inertia mass acceleration.

$V_{base}$ = Theoretical base shear.

$V_{cell}$ = Wall base shear recorded by the load cell mounted between the test wall and shaketable.

$V_e$ = Maximum elastic base shear.

$V_E$ = Design earthquake lateral load.

$V_{os}$ = Load offset for degrading stiffness $k_d$.

$V_o$ = Wall overstrength.

$V' P$ = Shear force in framing plate member.

$V' P'$ = Shear force in framing plate member at wall ultimate strength $V'$.

$V_s$ = Shear force in framing stud member, may be further subscripted $E$ for edge framing stud or $I$ for interior framing stud.

$V's$ = Shear force in framing stud member at wall strength $V'$, may be further subscripted $E$ for edge framing stud or $I$ for interior framing stud.
\( V_{ws} \) = Permissible wind/seismic wall lateral load.

\( V_y \) = Nominal yield load.

\( W_t \) = Total seismic weight of building.

\( X \) = Horizontal framing joint force, may be subscripted E for edge connection or I for interior connection.

\( X' \) = Horizontal framing joint force at the wall ultimate strength \( V' \), may be subscripted E for edge framing connection or I for interior connection.

\( X_o \) = Horizontal framing joint force at the wall overstrength \( V_o \), may be further subscripted E for edge framing stud or I for interior framing stud.

\( y(t) \) = Relative displacement.

\( \dot{y}(t) \) = Relative velocity.

\( \ddot{y}(t) \) = Relative acceleration.

\( \ddot{y}_g(t) \) = Shaketable acceleration.

\( Y_{M,max} \) = Position of maximum bending moment from end of framing stud.

\( Y_r = y/H \) = Vertical framing joint force, may be subscripted E for edge connection and I for interior connection.

\( Y' \) = Vertical framing joint force at wall ultimate strength \( V' \), may be subscripted E for edge connection and I for interior connection.

\( \ddot{Y}(t) \) = Absolute response acceleration of inertia mass.
\[ y_o = \text{Vertical framing joint force at the wall overstrength} \]
\[ z = \text{Effective modulus of framing at the critical section.} \]
\[ z_p = \text{Plastic Section Modulus.} \]
\[ z = \text{Position of nail plastic hinge.} \]
\[ a = H/B = \text{Aspect ratio of sheathing or Stress Block factor or Pinching Factor.} \]
\[ a_t = t_s/t_f = \text{Ratio of sheathing thickness to framing thickness.} \]
\[ \beta = \text{Stress block factor or softening parameter.} \]
\[ Y = \text{Shear strain, may be subscripted s for sheathing or f for framing.} \]
\[ \Delta_a = \text{Horizontal wall deflection arising from anchorage deformation.} \]
\[ \Delta_{av} = \text{Average nail slip along perimeter of sheathing.} \]
\[ \Delta_b = \text{Embedment of nail shank.} \]
\[ \Delta_{bs} = \text{Embedment of nail shank at the member surface.} \]
\[ \Delta_{c,x}, \Delta_{c,y} = \text{Separation of framing joints in x and y directions.} \]
\[ \Delta_{f} = \text{Deflection of framing stud member or shear wall deflection arising from axial strains in framing members.} \]
\[ \Delta_{hd} = \text{Vertical displacement of vertical anchorage connections.} \]
\[ \Delta_j = \text{Joint slip.} \]
\[ \Delta_k = \text{Lozenging wall deflection.} \]
\[ \Delta_{kn} = \text{Lozenging wall deflection arising from nail slip or Nail point withdrawal from framing member.} \]
\[ \Delta_{ls} = \text{Lozenging wall deflection arising from shear distortion of sheathing.} \]

\[ \Delta_{max} = \text{Maximum lateral deflection.} \]

\[ \Delta_{Ngap} = \text{Initial slackness in negative direction.} \]

\[ \Delta_o = \text{Yield embedment.} \]

\[ \Delta_{Pgap} = \text{Initial slackness in positive direction.} \]

\[ \Delta_p = \text{Nail slip at which ultimate nail strength } P' \text{ occurs.} \]

\[ \Delta_s = \text{Horizontal displacement of horizontal shear anchorage connection.} \]

\[ \Delta_t = \text{Time Step.} \]

\[ \Delta_T = \text{Total horizontal wall deflection.} \]

\[ \Delta_y = \text{Lateral deflection when first yield is reached.} \]

\[ \Delta_{x', y'} = \text{Nail slip in x and y directions.} \]

\[ \Delta'_{x', y'} = \text{Nail slip between the corner of the sheathing and the corner of the framing members in the x and y directions.} \]

\[ \theta = \text{Nail plastic hinge rotation or Angle of nail slip with respect to the longitudinal axes of the framing members.} \]

\[ \lambda = \frac{c}{c_r} = \text{Ratio of critical damping, may be subscripted s for equivalent damping ratio for the slackness oscillator.} \]

\[ \lambda = \frac{f_{b,2}}{f_{b,1}} = \text{Ratio of timber bearing strength for joint members 2 and 1.} \]

\[ \mu = \text{Mean, may be further subscripted p for sheathing nailing or F for non-ductile components.} \]

\[ \mu_t = \text{Coefficient of friction at contact surfaces between timber and timber.} \]
\( \mu_{ts} \) = Coefficient of friction at contact surfaces between timber and nail shank.

\( \mu_{\Delta} = \Delta_{\text{max}} / \Delta_y \) = Displacement ductility factor, may be further subscripted \( \text{EP} \) for elastoplastic idealisation, \( P \) for pinching idealisation or \( \text{PM} \) for modified pinching idealisation.

\( \nu \) = Poisson's ratio for sheathing.

\( \pi \) = 3.1416.

\( \rho \) = Timber density.

\( \rho_b \) = Timber Basic Density.

\( \sigma_t \) = Standard Deviation of the Ultimate Tensile Stress \( f_{\text{ut}} \).

\( \sum \) = Symbol denoting the summation of a variable.

\( \phi \) = \( d_n/t_s \).

\( \phi \) = Phase angle.

\( \phi_m \) = Bending moment reduction factor for framing stud member, may be further subscripted \( E \) for edge framing stud or \( I \) for interior framing stud.

\( \phi'_m \) = Bending moment strength parameter, may be further subscripted \( E \) for edge framing stud or \( I \) for interior framing stud or \( p \) for framing plate member.

\( \phi_o = P_o / P_E \) = Overstrength factor.

\( \phi_{o,m} = M_{o,s} / M_{o,s(\text{rigid})} \) = Bending moment overstrength reduction factor.

\( \phi_{o,x} = X_o / X_{o(\text{rigid})} \) = Framing joint overstrength reduction factor, may be further subscripted \( E \) for edge framing stud or \( I \) for interior framing stud.

\( \phi'_p \) = Framing plate strength parameter.
\[ \phi_s' = \text{Framing stud strength parameter.} \]

\[ \phi_t = \frac{f_{o,t}}{f_{o,t}(M,\text{max})} = 1 - \frac{y_{m,\text{max}}}{H} \]

\[ \phi_x = \text{Horizontal framing force reduction factor, may be further subscripted E for edge connection or I for interior connection.} \]

\[ \phi_x' = \text{Horizontal framing joint strength parameter, may be further subscripted E for edge connection or I for interior connection.} \]

\[ \phi_y' = \text{Vertical framing joint strength parameter.} \]

\[ \psi = \text{Bilinear factor.} \]

\[ \omega = \text{Timber moisture content.} \]

\[ \omega_g = \text{Excitation or shaketable frequency.} \]

\[ \omega_{\text{nat}} = \sqrt{\frac{k_0}{M}} = \text{Natural frequency.} \]

\[ \omega_s = \text{Spring stiffness frequency.} \]
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1

INTRODUCTION

1.1 INTRODUCTION

Plywood sheathed shear walls are commonly used to resist lateral loading arising from wind and earthquakes [1]. The shear wall consists of sheathing panels which are fastened with ductile sheathing nails to framing members around the perimeter. The strength and stiffness of the shear wall is achieved through the interaction of the sheathing panels and framing members. The constitutive elements form a potentially efficient earthquake resistant unit. For service conditions and during minor earthquakes that might occur more than once during the life of the structure, the rigidity of the sheathing and the high initial stiffness of the sheathing nailing limit lateral displacements to low values. During a major earthquake that might occur during the life of the structure, the ductile sheathing nailing permits large inelastic wall displacements while maintaining load capacity. Although the monotonic static load resistance of shear walls and nailed joints has been reported [2,3], there is little information available concerning their behaviour when subjected to static and dynamic reverse cyclic loading. Furthermore,
although shear walls have performed satisfactorily during previous earthquakes [4,5], this reputation has generally been based on the performance of light weight, symmetrical and structurally redundant buildings [6,7]. Their performance as primary seismic resisting elements is unproven in more modern structures which incorporate large inertia masses and little redundancy.

1.2 SEISMIC DESIGN PHILOSOPHY IN NEW ZEALAND

1.2.1 Objectives

The objective of modern loading codes [7,10] is for structures to remain elastic during minor earthquakes, but sustain, without collapse, large inelastic deformations during major earthquakes. Therefore, although the structure does not collapse, it may be severely damaged, and beyond repair.

The capacity design procedure [9] has been previously developed as part of the earthquake resistant design procedure for reinforced concrete structures to ensure that these structures do not collapse. This procedure requires that:

1. Preselected ductile elements are suitably detailed to permit large inelastic displacements.
2. Other parts of the structure are designed to be stronger than the ultimate strength of these ductile elements; thereby preventing premature failure or collapse of the structure.

As yet there is no capacity design methodology for plywood sheathed shear walls. Present knowledge of shear wall behaviour points to the sheathing nailing as the preferred ductile elements as a means of avoiding failure of the timber members in bending, shear, and tension. However, it is uncertain whether the sheathing nails are able to sustain the large displacement cycles imposed during a major earthquake.
12.2 **Loadings**

The NZS 4203:1984 Code of Practice for General Structural Design and Design Loadings for Buildings [10] specifies equivalent static lateral loads to determine the strength of the structure necessary for withstanding the design earthquake. The design earthquake load $V_E$ on a structure of weight $W_t$ is specified as $V_E = C R(SM) W_t$, where C is the basic seismic coefficient obtained from the NZS 4203:1984 inelastic design spectrum, and depends on site location, and the fundamental period of vibration of the structure; $R$ is the risk factor; and SM is the combined structural type and material term. Structures designed to the acceleration levels C given by the NZS 4203:1984 inelastic design spectrum must exhibit ductility if collapse is to be avoided during a major earthquake attack.

Ductility is the ability of a structure to undergo large reverse cycles in the inelastic range without substantial reduction in strength. A measure of the ductility capacity of the structure is the displacement ductility factor $\mu_\Delta = \Delta_y / \Delta_{\text{max}}$ where $\Delta_{\text{max}}$ is the maximum lateral deflection, and $\Delta_y$ is the lateral deflection when first yield is reached. NZS 4203:1984 requires that structures designed to the acceleration levels C given by the inelastic design spectrum, be capable of $\mu_\Delta = 4$.

In recognition that some structures possess displacement ductility greater than or less than $\mu_\Delta = 4$, the design earthquake load $V_E$ is modified by the structural type and material term SM. The intention of the SM term is to provide the structure with sufficient strength so that its displacement ductility capacity is not exceeded; thus avoiding collapse during a major earthquake attack. The SM terms assigned in NZS 4203:1984 correspond to $SM = 4/\mu_\Delta$.

Experimental studies are necessary to determine the appropriate displacement ductility capacity $\mu_\Delta$ and hence SM, for different types of structures (frames, shear walls, etc.) and materials.
(reinforced concrete, steel, etc). No such study has been completed for plywood sheathed shear walls. Rather, the SM = 1.0 value currently specified in Table 5(b) of NZS 4203:1984 for plywood sheathed shear walls is based on the results of only a few tests, and further studies are required to justify it.

1.3 SCOPE OF RESEARCH

The objective of this study was to develop a seismic design methodology for plywood sheathed shear walls based on the "capacity design procedure" [9], so that when shear walls are subjected to large displacement demands during a major earthquake they can be sustained without collapse. The capacity design procedure has long been used for the seismic design of structures built with other materials, but its application to timber structures and in particular plywood sheathed shear walls, has received little attention.

The load carrying and energy dissipating mechanisms of a plywood sheathed shear wall are examined in detail. Analytical models for predicting the elastic behaviour and ultimate strength of shear walls, and the ultimate strength of nailed sheathing joints, are formulated in order to develop the design procedure. An experimental investigation of nailed sheathing joints was undertaken. Special attention was given to their reverse cyclic behaviour when subjected to displacements well into their inelastic range. Complementary to these detailed investigations, quasi-static and shakeable testing of full scale shear walls was undertaken. Results from this testing enabled assessment of the design procedure, and validation of a theoretical time history single degree of freedom idealisation describing the dynamic response of shear walls.

The investigation is presented in the following order:

Chapter 2 describes an elastic and ultimate strength model predicting the behaviour of sheathed shear walls.
Chapter 3 examines sheathing nail behaviour. An ultimate strength model for nailed joints is described, and used to identify the most significant parameters that affect the strength of nailed sheathing joints. Experimental reverse cyclic behaviour of nailed sheathing joints is also reported, and their failure modes and inelastic displacement capacity are identified.

Chapter 4 introduces an experimental investigation of shear wall behaviour.

Chapter 5 presents the results of 4 quasi-static shear wall tests, 5 sinusoidal shaketable shear wall tests, and 3 simulated earthquake shaketable tests. The chapter includes a discussion of the observed behaviour and comparisons with theoretical predictions, and presents a seismic design methodology for sheathed shear walls.

In Chapter 6, a comparative study is made of the seismic behaviour of pinching load-deflection hysteresis loops (characteristic of shear walls), and elastoplastic load-deflection hysteresis loops.

Finally, Chapter 7 summarises the conclusions made at the end of each previous chapter and identifies future research needs.
2

PROPOSED ANALYTICAL MODELS FOR PLYWOOD SHEATHED SHEAR WALLS

2.1 INTRODUCTION

The structural analysis of timber shear walls and diaphragms is complex. Models for predicting the overall load-deflection behaviour of shear walls have been advanced in recent years, but few models predict the loads imposed on the components within the wall.

In the first part of this introduction, a simple diaphragm theory is reviewed. The theory is reviewed here in some detail because it leads to a basic understanding of the load carrying mechanism of the shear wall. Furthermore, the simple diaphragm theory forms the basis of the shear wall design expressions given in NZS 3603:1981; Code of Practice for Timber Design [11]. Following the description of the simple diaphragm theory, the more sophisticated closed form and finite element models predicting shear wall load-deflection behaviour are reviewed.
Fig. 2.1  Plywood sheathed shear wall. (a) Typical construction details. (b) Typical wall displacements.
Fig. 2.1  Plywood sheathed shear wall.  (a) Typical construction details.  (b) Typical wall displacements.
2.1.1 Simple Diaphragm Theory

Figure 2.1(a) shows a single panel wall of height $H$ and width $B$ in which it is assumed that the sheathing is rigid and fixed by flexible nail fasteners to pin-jointed framing members, and that nail forces acting in the direction perpendicular to the framing members are assumed to be zero. When subjected to a lateral load (wall load) $V$ acting along the top framing plate, the framing deforms to the shape shown in Fig. 2.1(b), and the sheathing slips over the framing. The relative movement between the sheathing and framing is accommodated by nail slip in directions parallel and perpendicular to the edge of the sheathing.

![Simple diaphragm theory diagram](image)

*Fig. 2.2 Simple diaphragm theory.*
2.1.1.1 Shear Wall Actions

With reference to Fig. 2.2, the applied load $V$ at the top of the wall is resisted by vertical anchorage forces $VH/B$ and horizontal anchorage force $V$ at the supports. The vertical anchorage force acting on the tension framing stud applies axial tension to the framing stud, and shear flow along the nail line transfers this axial tension into the sheathing edge. The shear transfer induces a triangular distribution of axial tension along the length of the tension framing stud having a maximum value of $VH/B$ at the base of the wall (see Fig. 2.2). Similarly, shear transfer along the length of the compression framing stud induces a triangular distribution of axial (compression) force, having a maximum value of $VH/B$ at the base of the wall. The shear flow transferred by the nail lines subjects the sheathing to pure shear. The resulting shear stress in the sheathing of thickness $t_s$ is $V/t_s$. Equilibrium of the top framing plate, shown in Fig. 2.2, requires each nail having spacing $c$ to carry the load $Vc/B$. There is no axial force in the top framing plate when the applied wall load $V$ is uniformly distributed along it, as illustrated in Fig. 2.2. The expression for the top framing plate force given in NZS 3603:Table 29 assumes that the load $V$ is applied at the end of the top framing plate.

When the compression framing stud butts into the inside face of the framing plate, as shown in Fig. 2.2, perpendicular-to-grain bearing stresses are induced in the bottom framing plate. Hold-down connections are required to resist the tensile vertical anchorage forces.

The interior framing studs shown in Fig. 2.1 assist in supporting any gravity loads on the shear wall, and stiffen the sheathing against buckling. In multi-panel shear walls, internal framing studs also provide shear splices between adjacent sheathing panels. Internal framing studs are not axially loaded by the lateral load.
Fig. 2.3 Terminology diagram for wall deflection.

2.1.1.2 Shear Wall Deflection

The total horizontal deflection $\Delta_T$ is the sum of wall lozenging deflection $\Delta_L$ and wall deflections due to anchorage displacements $\Delta_a$, as shown in Fig. 2.3.

Wall lozenging deflection is a result of nail slip and shear distortion of the sheathing. Lozenging deflections arising from each of these are derived separately, and then superimposed. The expression for shear wall deflection given in NZS 3603:Table 29 also includes a further term to account for the effects of axial strains in the framing members. However, this deflection term is normally very small compared to the nail slip, sheathing shear strain, and anchorage displacement terms.
(a) **Nail Slip**

Nail slip is the dominant cause of wall lozenging deflection. Figure 2.1(b) shows the lozenging of the framing due to nail slip alone, i.e., the sheathing is assumed to be rigid. The shear strain $\gamma_f$ of the framing is given by $\gamma_f = \gamma_1 + \gamma_2$, where $\gamma_1 = 2\Delta'_y/B$ and $\gamma_2 = 2\Delta'_x/H$ and $\Delta'_x$ and $\Delta'_y$ define horizontal and vertical nail slip at the corner of the panel, as shown in Fig. 2.1(b). For constant load $Vc/B$ in each nail along each framing member then $\Delta'_x = \Delta'_y = \Delta_{av}$, and $\gamma_f = 2\Delta_{av}(1/B + 1/H)$, and the lozenging deflection $\Delta_{ln}$ of the wall arising from nail slip is $\Delta_{ln} = 2\Delta_{av}(1+\alpha)$, where $\alpha = H/B$ is the aspect ratio of the sheathing panel. The aspect ratio $\alpha$ has been taken for convenience as unity in the NZS 3603:Table 29 [11] expression for shear wall deflection. The average nail slip $\Delta_{av}$ is determined from the load-slip characteristics of the sheathing nails for specified nail load $Vc/B$.

(b) **Sheathing Shear Strains**

Further lozenzing of the wall arises from shear distortion of the sheathing. The shear strain $\gamma_s$ of the sheathing is $\gamma_s = V/BGt_s$, where $G$ is the modulus of rigidity of the sheathing. The lozenging deflection $\Delta_{ls}$ of the wall due to sheathing strain is $\Delta_{ls} = VH/BGt_s$.

(c) **Anchorage Displacements**

Distortion of the horizontal shear connection results in rigid translation of the wall $\Delta_s'$, as shown in Fig. 2.3. Vertical displacements $\Delta_{hd1}$ and $\Delta_{hd2}$ (defined in terms of positive upwards sign convention) of the vertical anchorage connections located at the bottom corners of the wall cause a rigid body rotation of the wall, resulting in a horizontal deflection at the top of the wall of $H/B$ ($\Delta_{hd1} - \Delta_{hd2}$) (see Fig. 2.3). Horizontal wall deflection $\Delta_a$ arising from the anchorage displacements is then
\[ \Delta_a = \Delta_s + \frac{H}{B}(\Delta_{hd1} - \Delta_{hd2}) \]

(d) **Framing Axial Strains**

The triangular distribution of axial force along each framing stud member (see Fig. 2.2) results in corresponding axial strains, and these strains cause horizontal deflection \( \Delta_f \) at the top of the wall. This deflection can be idealised as being the deflection of a cantilever 'I' beam of length \( H \) in which the framing studs act as beam flanges and the sheathing as the web. The lateral flexural deflection \( \Delta_f \) at the top of the wall is then \( \Delta_f = \frac{VH^2}{3EI} \), where the flexural rigidity \( EI \) can be approximated as \( EI = \frac{1}{2}EAH^2 \) and where \( E \) and \( A \) are the modulus of elasticity and area of the edge framing stud members.

<table>
<thead>
<tr>
<th>Wall Load</th>
<th>Sheathing stress</th>
<th>axial stress</th>
<th>Deflection at top of Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V = \frac{PB}{C} )</td>
<td>( f_s = \frac{V}{Bt_s} )</td>
<td>( f_t = \frac{2V}{A} )</td>
<td>0</td>
</tr>
</tbody>
</table>

\( \Delta_a = \Delta_s + \frac{H}{B}(\Delta_{hd1} + \Delta_{hd2}) \)
2.1.2. A Survey of Literature Regarding the Analysis of Plywood Sheathed Shear walls

Early investigators [12, 13, 14] carried out experimental tests on shear walls incorporating standard 2.4m by 1.2m panels and constant sheathing nail spacing. They advanced empirical equations showing that wall stiffness and strength were highly dependent on the number and strength of the sheathing nails.

In 1976 Burgess [15] described an energy method to obtain a closed form solution for wall stiffness. A similar derivation was later published by Tuomi and McCutcheon [16]. The theory was based on the assumed in-plane wall displacements shown in Fig. 2.4(a). When horizontal load is applied along the top framing plate, the framing deforms into a parallelogram and the sheathing nails deform permitting the sheathing to slip over the framing. In the Burgess model, the in-plane rotation of the sheathing over the framing is constrained so that $\gamma_1 = \gamma_2$ (Fig. 2.4(a)). The model imposes no restriction on panel aspect ratio, but is valid only for equally spaced sheathing nails. In the derivation, Burgess assumed that the load-slip behaviour of the sheathing nails is linear, and the sheathing and framing remain rigid. Having defined the deformed shape of the wall, Burgess showed that nail forces act in directions parallel and perpendicular to the framing members, as seen in Fig. 2.4(b). This theory is, therefore, more rigorous than the simple diaphragm theory (Fig. 2.2) as the latter neglects nail forces acting in the direction perpendicular to that of the framing members. Nevertheless, Burgess showed that the overall wall stiffness predicted by the simple diaphragm theory was similar to that obtained using his more rigorous theory.
Fig. 2.4 Shear wall with rigid framing members. (a) Wall displacements. (b) Force distribution in framing members.
In 1985, McCutcheon [17] extended the Tuomi - McCutcheon model to include shear distortion of the sheathing, and nonlinear load-slip behaviour of the sheathing nails. This model showed good agreement with test results up to shear wall deflections (in terms of Wall height H) of approximately H/160.

Kuenzi [18] proposed an energy method to produce a more general solution to the Burgess model by allowing γ₁ and γ₂ to be unconstrained (Fig. 2.4(a)). This method was presented again by Walker [19] in 1978 formulated in terms of the force equilibrium method.

In 1982 Easley, Poohani and Dodds [20] presented a mixed force and displacement approach to derive a closed form nonlinear shear load-deflection relationship for shear walls. They observed that the vertical edges of the sheathing and framing of sheathed metal shear walls rotated equally such that the nail slip in the direction perpendicular to the framing studs was zero, i.e. γ₂ = 0 in Fig. 2.4(a). While this might be the case for very slender panels, it is inconsistent with the measured sheathing slip in recent tests [33 and 34] of plywood and particle board sheathed panels having the usual aspect ratio α = H/B = 2. Clearly, the assumption of no nail slip in the direction perpendicular to the framing stud γ₂ = 0 may be justified if the framing members are not connected at their ends, such that the framing studs are free to separate from the framing plates. However, this is not normally the case. Although a nonlinear load-slip relationship was used for determining the nail forces acting in the direction parallel to the framing members, a linear elastic relationship was assumed for the nail forces acting in the direction perpendicular to the framing plates, with a stiffness equal to the secant stiffness of the actual nonlinear response. Easley, et. al. found good agreement between their shear wall formula and the results of full scale shear wall tests and nonlinear finite element analysis. Comparisons were made up to horizontal deflections of H/240.
In 1985 Gupta and Kuo [21] advanced a more rigorous, although not closed form, load-deflection model for shear walls. Gupta and Kuo observed that the nail forces acting in the direction perpendicular to the framing members caused the framing studs to deform, as shown in Fig. 2.5(a). The deformed framing stud reduces the relative movement between the framing stud and sheathing (i.e. nail slip) in the direction perpendicular to the framing stud. This in turn results in nail forces acting in the direction perpendicular to the framing stud that are lower than those nail forces indicated in Fig. 2.4(b) for the rigid framing case. This reduction in nail forces is consistent with the nail force distribution given by the finite element analysis reported by Easley, et. al. [20]. Results from the Gupta and Kuo model were shown to be in good agreement with test results up to wall displacements of H/160. In addition, Gupta and Kuo showed that the overall load-deflection behaviour of the wall was not significantly influenced by framing stud flexibility.

In 1984 Dean, Stewart and Moss [22] described a simple shear transfer approach to analyse shear walls with openings. The method has particular merit in the design office where the loads on wall components, especially the highly stressed components around openings, can be easily assessed.

In 1985, Itani, Tuomi and McCutcheon [23] extended the Tuomi-McCutcheon model to determine the strength of continuous wall panels with and without openings.

In parallel with the development of these shear wall formulas, Foschi [24] and Itani and Cheung [25] independently advanced general finite element models for the static nonlinear analysis of diaphragms and shear walls. In the finite element analysis, the sheathing is modelled by linear elastic two dimensional plane stress elements, and the framing by linear elastic beam elements. The sheathing nails are represented by nonlinear elastic springs. Results from the finite element analysis have agreed well with test results up to wall displacements of H/160.
The finite element models are suited to predicting the nonlinear load-deflection behaviour of diaphragms and shear walls having any geometry, but they lack the simplicity of the closed form solutions, and are not as suitable for design office use. In contrast, the existing closed form equations for shear walls do not provide a method for designing the wall components to resist the applied internal forces.

2.1.3 Scope of this Chapter

In this chapter, models for predicting in-plane stiffness and ultimate strength of shear walls are developed. The proposed models are particularly suited for design office use. Sheathing nail loads, framing and sheathing stresses, framing joint forces, and anchorage forces are easily calculated. The results from the proposed models are compared in Chapter 4 with previous experimental data and finite element analyses.

2.2 Assumptions

In deriving the equations for the proposed elastic shear wall model and the ultimate strength model, the following common assumptions are made:

1. The sheathing material is rigid.
2. The framing members behave linear elastically.
3. The framing plate members are assumed to be rigid.
4. Framing stud members have the same cross-sectional area, and are pin-jointed to the framing plate members.
5. The sheathing nails are uniformly spaced around the perimeter of the sheathing.
6. Nail spacing $c$ is assumed to be sufficiently small for the shear connection between the sheathing and framing to be considered continuous.
7. The relative movement of the sheathing over the framing members is as illustrated in Fig. 2.5(a), and $\gamma_1$ and $\gamma_2$ are independent and non-zero.

2.3 ELASTIC SHEAR WALL MODEL

2.3.1 General

In this section, the elastic analysis of timber sheathed shear walls advanced by Walker [19] is extended to include flexible framing members. The Walker theory was adopted for its simplicity and suitability for design office use. Furthermore, the assumed in-plane slip of the sheathing over the framing is consistent with measured sheathing slip during tests [33, 34].

In deriving the following equations for elastic shear wall behaviour, the sheathing nails are assumed to behave linear elastically with stiffness $k$.

Figure 2.4(a) shows a single panel wall of height $H$ and width $B$. The wall load $V$ along the top framing plate is resisted by vertical anchorage forces $VH/B$ and horizontal anchorage forces $V$ at the supports. Figure 2.4(b) shows the nail forces acting on the framing for the case when the framing is rigid. The nail forces acting in a direction perpendicular to the framing, and which were neglected in the simple diaphragm theory, cause:
(a) internal framing joint forces $X$ and $Y$ (Fig. 2.4(b)), and
(b) flexural stresses in the framing.

For shear wall components of usual proportions the framing studs are relatively flexible, and the nail forces perpendicular to the framing will cause the framing studs to bend, as observed by Gupta and Kuo [21], and as shown in Fig. 2.5(a). Lower deflections develop in the framing plates, which are shorter than the framing studs and are stiffened by the adjacent floor beams. Finite element analysis of a 3 panel 2.4 x 3.6 m long shear wall [20] indicated that the framing plates were effectively rigid. The framing plates are assumed to be rigid throughout this study.
Fig. 2.5 Shear wall with flexible framing stud members. (a) Wall displacements. (b) Deformed shape of right hand framing stud member. (c) Force distribution in framing members.
Figure 2.5(c) shows the nail forces acting on flexible framing studs. The nail forces acting in the direction perpendicular to the framing stud are substantially less than those indicated for rigid framing studs (Fig. 2.4(b)), and this leads to reduced flexural stresses and framing joint forces.

2.3.2 Framing Behaviour

2.3.2.1 In-Plane Lateral Deflection of Framing Stud Member

Figure 2.5(b) shows a framing stud member of height $H$ and having in-plane lateral deflection $\Delta_f$. The deflected shape of the framing stud member in terms of co-ordinates $x$ and $y$, is assumed to be represented by a Fourier Series of the form,

$$\Delta_f = \sum_{i=2,4,6,...}^\infty C_i \sin \left(\frac{i\pi y}{H}\right)$$

(2.1)

Where $C_i$ are constants.

For a specified horizontal slip $\Delta'_x$ between the corner of the sheathing and the corner of the framing members (see Fig. 2.5(a)), the deflection of the framing stud member can be found using energy methods. The framing stud member will take up the deflected shape that will minimise the total strain energy arising from the nail forces causing the framing stud member to bend, and the bending of the framing stud member itself. Nail slip in the direction parallel to the framing stud does not contribute to the strain energy associated with (small deflection) bending of the framing stud member, and therefore is not included in the energy equations.

The corresponding bending strain energy for a single framing stud member is
\[ U_{fx} = \frac{1}{2} \left[ \frac{H}{2} \right] EI \left[ \int \frac{d^2 \Delta f}{dy^2} \right] \left[ -\frac{H}{2} \right] \]  

(2.2)

Substituting in the assumed deflected shape of the framing stud members (Eq. 2.1), and integrating Eq. 2.2 gives

\[ U_{fx} = \frac{EI \pi^4}{4H^3} \sum_{i=2,4,6, \ldots}^\infty \frac{4C_i^2}{i^4} \]  

(2.3)

The nail slip at any location is the relative movement between the framing members and sheathing edge. Nail slips in the horizontal and vertical directions are denoted by \( \Delta_{x,j} \) and \( \Delta_{y,j} \) respectively, as shown in Fig. 2.5(a), where \( j = 1,2,3, \ldots n \) and \( n \) is the total number of nails around the sheathing perimeter. For nail \( j \) having stiffness \( k_j \), the strain energy \( U_{nx,j} \) in the direction perpendicular to the framing stud member is

\[ U_{nx,j} = \int_0^{\Delta_{x,j}} k_j d\Delta_x = \frac{1}{2} k_j \Delta_{x,j}^2 \]

where the horizontal slip \( \Delta_x \) between the edge of the sheathing and framing stud member (see Fig. 2.5(b)) is

\[ \Delta_x = \frac{2\Delta_y}{H} - \sum_{i=2,4,6, \ldots}^{\infty} C \sin(i\pi y/H) \]  

(2.4)

Assuming the nail spacing \( c \) is sufficiently small for the shear connection to be considered as continuous along the framing to sheathing interface and \( k_j = k \), then the strain energy \( U_{nx}(y) \) per unit length of the shear connection in the direction perpendicular to the framing stud member is
\[ U_{nx}(y) = \frac{k}{2c} \Delta_x^2 \]  

(2.5)

The total strain energy \( U_{nx} \) of the shear connection along the length of the framing stud member follows by integrating Eq.2.5 with respect to \( y \)

\[ U_{nx} = \sum_{i=2,4,6,...}^{\infty} \frac{k}{2c} \left( \frac{C_i H}{2} - \frac{4\Delta_x^2 H C_i}{1\pi} \frac{(-1)^{i/2}}{i\pi} + \frac{\Delta_x^2 H}{3} \right) \]  

(2.6)

The total strain energy \( U_x \) in the direction perpendicular to the framing stud member is \( U_x = U_{fx} + U_{nx} \). Minimising the total strain energy with respect to \( C_i \), gives

\[ \frac{dU_x}{dC_i} = \frac{EI\pi^4}{4H^3} \frac{4}{2i} C_i + \frac{kC_i H}{2c} - \frac{4k\Delta_x H}{2i\pi c} \frac{(-1)^{i/2}}{i\pi} = 0 \]  

(2.7)

and solving for \( C_i \) we find

\[ C_i = \frac{4\Delta_x^2 (-1)^{i/2}}{1\pi(K_e i^4 + 1)} \]  

(2.8)

where \( K_e \) is an elastic stiffness parameter defined as

\[ K_e = \frac{C_E I \pi^4}{H^4 k} \]  

(2.9)

The deflected shape of the framing stud is determined from Eqs. 2.1 and 2.8 by taking as many terms in the series as is necessary.
to obtain the required accuracy. The position $y_{\text{max}}$ of the maximum framing stud deflection $\Delta_f,\text{max}$ is determined from Eq. 2.1 as

$$\frac{d\Delta_f}{dy} = 0 = \sum_{i=2,4,6,...}^{\infty} C_i \frac{i\pi}{H} \cos(i\pi y/H)$$

Solving for $y_{\text{max}}$ the maximum framing stud deflection $\Delta_f,\text{max}$ from Eq. 2.1 is plotted in Fig. 2.6 with respect to the elastic stiffness parameter $k_e$.

**Fig. 2.6** The relationship between the elastic stiffness parameter $k_e$ and maximum framing stud deflection $\Delta_f,\text{max}$. 

(2.10)
Fig. 2.7 Force distribution, deflected shape, moment diagram, shear force diagram and axial force diagram for right hand (tension) framing stud member.

2.3.2.2 Framing Member Joint Forces

Figure 2.7 shows the free body diagram of the right hand tension framing stud member, and the corresponding shear force, bending moment, and axial force diagrams. Horizontal framing joint force $X$ acts at each end to maintain equilibrium with the nail forces acting in the direction perpendicular to the framing stud member (see Fig. 2.7). Moment equilibrium of the forces acting on the framing stud member about the origin $O_s$ gives

$$
\frac{H}{2} - \frac{k\Delta_x}{c} \int y \, dy - XH = 0
$$

(2.11)
Substituting Eq. 2.4 into Eq. 2.11, and solving for the horizontal framing joint force \( X \) gives

\[
\phi_x \frac{k \Delta'H}{X} = \frac{X}{6c} \tag{2.12}
\]

where the horizontal framing joint force reduction factor \( \phi_x \) is defined as

\[
\phi_x = \frac{X}{X_{\text{rigid}}} = 1 - 6 \sum_{i=2,4,6,\ldots} \frac{4}{i^2 \pi^2 (K_e^4 + 1)} \tag{2.13}
\]

where \( X_{\text{rigid}} \) is the horizontal joint force (Fig. 2.4(b)) determined when the framing stud member is rigid. The framing force reduction factor \( \phi_x \) is plotted in Fig. 2.8 with respect to the elastic stiffness parameter \( K_e \).

Similarly, the vertical framing joint force \( Y \) can be determined from moment equilibrium of the forces acting on the top (rigid) framing plate member. Moment equilibrium about the origin \( O_p \) (See Fig. 2.4(b)), gives

\[
Y = \frac{k \Delta'B}{6c} \tag{2.14}
\]

### 2.3.2.3 Framing Member Axial Force

The axial force \( P_s(y_n) \) at any point \( y_n \) on the right hand (i.e. tension) framing stud member (\( y_n \) positive upwards from \( O_s \), Fig. 2.7) is

\[
P_s(y_n) = Y + \frac{k \Delta'}{c} \left( \frac{H}{2} - y_n \right) \tag{2.15}
\]

The maximum axial force \( P_{s,\text{max}} \) occurs at \( y_n = -H/2 \) and substitution of Eq. 2.14 into Eq. 2.15 gives
Fig. 2.8 The framing joint force reduction factor $\phi_x$ and bending moment reduction factor $\phi_m$ plotted against the elastic stiffness parameter $K_e$. 
\[ p_{s, \text{max}} = \frac{k \Delta' B}{c} \left( \alpha + \frac{1}{6} \right) \]  

(2.16)

where \( \alpha = H / B \) is the panel aspect ratio

Where the external load \( V \) is uniformly distributed along the top framing plate member (Fig. 2.4(a)), the axial force \( p(x_n) \) at location \( x_n \) from the origin \( O_p \) is

\[ p(x_n) = + \frac{\alpha k \Delta' \phi x x_n}{3c} \]

The maximum axial force occurs at \( x_n = \pm B/2 \) giving

\[ p_{\text{max}} = x = \pm \frac{\phi k \Delta' H}{6c} \]

(2.17)

2.3.2.4 Framing Member Bending Moment

Moment equilibrium about position \( y_n \) gives (see Fig. 2.7) the expression for the bending moment \( M_s \) in the framing stud member

\[ M_s(y_n) = X \left( \frac{H}{2} - y_n \right) - \int_{y_n}^{H/2} p_x(y)(y - y_n) \, dy \]

(2.18)

where \( p_x(y) = \frac{k \Delta}{c} = \frac{k}{c} \left( \frac{2 \Delta'}{H} - \sum_{i=2,4,6..}^{\infty} C_i \sin \left( \frac{i y}{H} \right) \right) \) is the nail force per unit length acting in the \( x \) direction.

Then in terms of \( y_r = y_n / H \)

\[ M_s(y_r) = \frac{H^2 k \Delta'}{c} \left\{ \frac{\phi}{6} \left( \frac{1}{2} - y_r \right) - \left( \frac{1}{12} - \frac{y_r}{4} + \frac{y_r^3}{3} \right) \right. \]

\[ - \sum_{i=2,4,6..}^{\infty} \frac{4}{\pi^2 (i \pi^2 + 1) (k \Delta' i^2 + 1)} \left( y_r - \frac{1}{2} \right) + \left. \frac{\sin(i \pi y_r)}{i \pi (-1)^{i/2} + 1} \right\} \]

(2.19)
Bending moment \( M_s(y_r) \) is a maximum when

\[
\frac{dM_s(y_r)}{dy_r} = 0 = \frac{3}{12} - \frac{\phi}{6} - \frac{1}{4} \sum_{i=2, 4, 6, \ldots}^{\infty} \frac{4}{\pi^2 i^2 (k_e i^4 + 1)} \left( 1 + \frac{\cos(i\pi y_r)}{\frac{i}{2} + 1} \right) \tag{2.20}
\]

From Eqs. 2.19 and 2.20 the maximum bending moment \( M_s(y_r, \text{max}) \) in the framing stud member is

\[
M_s(y_r, \text{max}) = M_{s, \text{max}} = \frac{0.064\phi_m B^2 k\Delta_y}{c} \tag{2.21}
\]

where \( \phi_m \) is the bending moment reduction factor defined as

\[
\phi_m = \frac{M_{s, \text{max}}}{M_{s, \text{max}(\text{rigid})}} = \frac{1}{0.016} \left( \phi \left( \frac{1}{2} - y_r, \text{max} \right) - \frac{1}{12} \frac{Y_{r, \text{max}}}{4} + \frac{Y_{r, \text{max}}^3}{3} \right) - \sum_{i=2, 4, 6, \ldots}^{\infty} \frac{4}{\pi^2 i^2 (k_e i^4 + 1)} \left( y_r, \text{max} - \frac{1}{2} + \frac{\sin(i\pi y_r, \text{max})}{i\pi (\frac{i}{2} + 1)} \right) \tag{2.22}
\]

and \( M_{s, \text{max}(\text{rigid})} \) is the maximum bending moment that would occur if the framing stud member was rigid. The bending moment reduction factor \( \phi_m \) is plotted in Fig. 2.8 with respect to the elastic stiffness parameter \( k_e \). Figure 2.9 shows the bending moment distribution along the top half of the framing stud member for various values of the elastic stiffness parameter \( k_e \). The bending moments \( M_s \) in Fig. 2.9 have been normalised with respect to the maximum bending moment \( M_{s, \text{max}} \) (Eq. 2.21).

Similarly, the maximum bending moment \( M_{p, \text{max}} \) in the framing plate member is

\[
M_{p, \text{max}} = \frac{0.016B^2 k\Delta_y}{c} \tag{2.23}
\]
Fig. 2.9 Framing stud bending moment $M_s$ plotted against framing stud height $y_r = y/H$ for values of elastic stiffness parameter $k_e$.

2.3.2.5. Framing Member Shear Force

With reference to Fig. 2.7 the shear force $V_s(y_n)$ in the framing stud member at location $y_n$ is

$$V_s(y_n) = X - \int_{y_n}^{y_n} \frac{H}{2} k \Delta' \frac{x}{c} dy$$

(2.24)

Substituting Eqs 2.4 and 2.12 into Eq. 2.24 gives the shear force $V_s(y_n)$ at $y_n$ as
\[ v_s(y_r) = \frac{k\Delta'H}{c} \left( y_r^2 - \frac{3}{12} + \frac{4}{6} x + \sum_{i=2,4,6,...}^{\infty} \frac{4}{i^2 \pi^2 (x_i^4 + 1)} \left( \frac{\cos(i\pi y_r)}{(-1)^i + 1} \right) \right) \]  

(2.25)

where \( y_r = y_n / H \).

Similarly the shear force \( v_p(x_n) \) at location \( x_n \) on the framing plate member is

\[ v_p(x_n) = \frac{k\Delta'B}{y} \left( x_n^2 - \frac{1}{12} \right) \]  

(2.26)

where \( x_r = x_n / H \)

2.3.3. **Nail Slip Ratio** \( r_s = \Delta'_y / \Delta'_x \)

With reference to Fig. 2.5(a), consider nail \( j \) on the top framing plate member having position \( x_j, y_j \) with respect to the panel origin \( O \). The nail slip \( \Delta'_{x,j}, \Delta'_{y,j} \) at nail \( j \) is

\[ \Delta'_{x,j} = \Delta'_x \]  

(2.27)

\[ \Delta'_{y,j} = \frac{2\Delta'_y}{B} \]  

(2.28)

The nail slip \( \Delta'_{x,j}, \Delta'_{y,j} \) at nail \( j \) on the framing stud member at position \( x_j, y_j \) is

\[ \Delta'_{x,j} = \frac{2\Delta'_y}{H} - \sum_{i=2,4,6,...}^{\infty} C_i \sin(i\pi y_j / H) \]  

(2.29)

\[ \Delta'_{y,j} = \Delta'_{y} \]  

(2.30)
For nail \( j \) having stiffness \( k \), the nail force components \( P_{x,j} \) and \( P_{y,j} \) in the \( x \) and \( y \) directions are

\[
P_{x,j} = k \Delta_{x,j} \quad \text{and} \quad P_{y,j} = k \Delta_{y,j}
\]

(2.31)

Moment equilibrium of the nail forces acting on the sheathing panel free body about the origin (Fig. 2.5(a)) requires

\[
\begin{align*}
\sum_{\text{Nails}} & (k\Delta_{x,j} y_j - k\Delta_{y,j} x_j) + \sum_{\text{Nails}} (k\Delta_{x,j} y_j - k\Delta_{y,j} x_j) = 0 \quad (2.32)
\end{align*}
\]

Substituting Eq. 2.27 to 2.30 into Eq. 2.32 gives,

\[
\begin{align*}
\frac{2k}{c} & \left[ \int_{-\frac{B}{2}}^{\frac{B}{2}} (\Delta'_x y - \frac{2\Delta'_x}{B} x^2) dx + \right. \\
& \left. \int_{-\frac{H}{2}}^{\frac{H}{2}} \left( \frac{2\Delta'_x}{H} y^2 - \sum_{i=2,4,6,\ldots} \sum_{\text{BC and DA}} C_{i,\text{BC}} y \sin(i\pi y/H) - \Delta'_x y \right) dy + \right. \\
& \left. \int_{-\frac{H}{2}}^{\frac{H}{2}} \left( \frac{2\Delta'_x}{H} y^2 - \sum_{i=2,4,6,\ldots} \sum_{\text{BC and DA}} C_{i,\text{DA}} y \sin(i\pi y/H) - \Delta'_y y \right) dy = 0 \quad (2.33) \right.
\end{align*}
\]

and solving for the nail slip ratio \( r_s \)

\[
\begin{align*}
\frac{\Delta'_y}{\Delta'_x} &= \frac{3 + \alpha \phi_x}{1 + \alpha} \quad (2.34)
\end{align*}
\]

2.3.4 **Maximum Sheathing Nail Force**

The maximum nail force \( P_{j,\text{max}} \) occurs at the corners of the panel, and is given by

\[
P_{j,\text{max}} = k (\Delta'_{x}^2 + \Delta'_{y}^2)^{\frac{1}{2}}
\]

(2.35)
and substituting the nail slip ratio \( r_s = \Delta' / \Delta'_x \) into Eq. 2.35 gives

\[
P_{j,max} = k \Delta'_x \left( 1 + r_s \right)^{1/2}
\] (2.36)

### 2.3.5 Shear Wall Lateral Load \( V \)

Equilibrium of the top framing plate member \( AB \), (Fig. 2.5(c)) requires the wall load \( V \) of a single panel wall to be

\[
V = \frac{B k \Delta'_x}{c} + 2X
\] (2.37)

Substituting horizontal framing joint force (Eq. 2.12) into Eq. 2.37 gives

\[
V = \frac{B k \Delta'_x}{c} \left( 1 + \frac{\alpha}{3} \phi_x \right)
\] (2.38)

In terms of the maximum nail force \( P_{j,max} \) from Eqs. 2.36,

\[
V = \frac{\left( 1 + \frac{\alpha}{3} \phi_x \right) B P_{j,max}}{(1 + r_s^2)^{1/2}}
\] (2.39)

The dimensionless parameter \( cV/B P_{\text{max}} \) is tabulated in Table 2.2 for shear walls of different sheathing aspect ratio \( \alpha \) and framing stud member stiffness parameter \( K_s \). Wall load \( V \) is evidently insensitive to sheathing aspect ratio \( \alpha \) and framing stud flexural stiffness.
Table 2.2 Wall Load $V$ in Terms of Maximum Nail Force $P_{j,max}$.

<table>
<thead>
<tr>
<th>$K_e$</th>
<th>$\alpha$</th>
<th>$x_s$</th>
<th>$\frac{cV}{BP_{j,max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.5</td>
<td>0.70</td>
<td>0.96</td>
</tr>
<tr>
<td>=</td>
<td>1.0</td>
<td>1.00</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>1.43</td>
<td>0.96</td>
</tr>
<tr>
<td>0.05</td>
<td>0.5</td>
<td>0.67</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.91</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>1.23</td>
<td>0.90</td>
</tr>
<tr>
<td>0.0005</td>
<td>0.5</td>
<td>0.62</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.80</td>
<td>0.83</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0.97</td>
<td>0.81</td>
</tr>
</tbody>
</table>

Fig. 2.10 Shear stresses in the sheathing material.
2.3.6 Sheathing Stresses

Figure 2.10 shows the actual distribution of nail shear applied to the panel edges by the framing members. Shear is transferred in directions parallel and perpendicular to each edge. Neglecting the shear transfer perpendicular to the edges as small, the average shear stress $f_s$ in the sheathing is

$$f_s = \frac{V}{t_B s}$$  \hspace{1cm} (2.40)

2.3.7 Shear Wall Deflection Due to Nail slip

The lozenging of the framing shown in Fig. 2.4(a) is due to nail slip alone.

The shear strain $\gamma_f$ of the lozenged framing is $\gamma_f = \gamma_1 + \gamma_2$, where $\gamma_1 = 2\Delta'_x/H$ and $\gamma_2 = 2\Delta'_y/H$. The horizontal wall deflection arising from nail slip $\Delta_{ln}$ is

$$\Delta_{ln} = H\gamma_f = H \left( \frac{2\Delta'_x}{H} + \frac{2\Delta'_y}{B} \right)$$  \hspace{1cm} (2.41)

Substituting Eq. 2.34 into Eq. 2.41 gives

$$\Delta_{ln} = 2\Delta' (1 + \alpha r_s)$$

where $\alpha = H/B$ is the aspect ratio of the sheathing panel. The average nail slip $\Delta_{av}$ is defined here as the average nail slip around the perimeter, parallel to the panel edges. Thus

$$\Delta_{av} = \frac{B\Delta'_x + H\Delta'_y}{B + H} = \frac{\Delta'_x (1 + \alpha r_s)}{1 + \alpha}$$  \hspace{1cm} (2.42)

The horizontal deflection $\Delta_{ln}$ in terms of the average nail slip is then
\[
\Delta_{\text{av}} = 2 \left(1 + \alpha\right) \Delta_{\text{av}}
\]

(2.43)

The average nail force \( P_{\text{av}} \) corresponding to the average nail slip is (from Eq. 2.42)

\[
P_{\text{av}} = k \Delta_{\text{av}} = \frac{(1 + \alpha \frac{r_s}{3}) k \Delta'_s}{1 + \alpha}
\]

(2.44)

or in terms of wall load \( V \) (Eq. 2.38).

\[
P_{\text{av}} = k \Delta_{\text{av}} = \frac{1 + \alpha \frac{r_s}{3}}{(1 + \frac{r_s}{3}) (1 + \alpha)} \frac{Vc}{B}
\]

(2.45)

### Table 2.3 Equations for Single Panel Shear Walls.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Equation</th>
<th>Typical range limits</th>
<th>Nailer theory</th>
<th>Simple diagonal theory</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall load ( V ) (in terms of maximum nail force ( P_{\text{f,max}} ))</td>
<td>( V )</td>
<td>0.055V - 0.152V</td>
<td>0.2V</td>
<td></td>
</tr>
<tr>
<td>Anchorage forces</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>- horizontal</td>
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<tr>
<td>- vertical</td>
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<tr>
<td>Framing Plate Member</td>
<td></td>
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</tr>
<tr>
<td>- axial force</td>
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<tr>
<td>- bending moment</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>- shear force</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>Framing Stud Member</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- axial force</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>- bending moment</td>
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<tr>
<td>- shear force</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Framing joint forces</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- horizontal</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- vertical</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sheathing shear stress</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nail slip ratio ( \Delta_{\text{av}}/\Delta_{\text{av}} )</td>
<td>( \bar{r}_s )</td>
<td>0.97 - 1.23</td>
<td>1.43 - 1.60</td>
<td></td>
</tr>
</tbody>
</table>
2.3.8 **Equations for a Single Panel Wall**

In this section the equations derived for framing axial force, bending moment, shear force and joint forces are presented in terms of the wall load $V$. Only equations for shear walls having an aspect ratio $a = 2$ are presented here. Equations for shear walls incorporating other aspect ratios can be derived in a similar manner.

Table 2.3 compares the proposed equations with the corresponding Walker theory [19] equations, and the simple diaphragm theory equations. The Walker theory and the simple diaphragm theory represent two special cases of the more general proposed theory. In the Walker theory, the framing members are assumed to have infinite flexural stiffness. In the simple diaphragm theory, the nail forces acting in the direction perpendicular to the framing members are neglected, and this is the same as assuming that the framing members have zero flexural stiffness. Equations in Table 2.3 are compared over the range of elastic stiffness parameter $K_e$ typical of shear wall components of usual proportions (see Table 2.7).

Table 2.3 shows that wall load $V$ (in terms of maximum nail force $P_{j,\text{max}}$) predicted by the Walker theory and simply diaphragm theory is similar to that given by the more general theory, clearly showing that wall load $V$ is not overly sensitive to framing member flexibility. The bending moments and joint forces given by the proposed equations lie between the extreme values predicted by the Walker theory and the simple diaphragm theory. Framing stud member bending moment and joint force are shown to be particularly sensitive to the flexural stiffness of the framing stud member.
Fig. 2.11 External and Internal actions in a multi-panel shear wall.

2.3.9 Application to Multi-Panel Walls

Figure 2.11 shows a wall having n panels of width B, and height H. Each sheathing panel is assumed to be nailed to the framing members only. The sheathing panels themselves are not joined to each other along their vertical edges. Each panel is, therefore, free to slip over the framing without constraint by the adjacent panels. Equilibrium of the top framing plate member (see Fig. 2.11) requires the wall load V of a multi-panel wall to be

\[ V = \frac{nBk\Delta'}{c} + 2X_E + \sum_{k=1}^{\infty} 2X_I \]  

(2.46)

where \( X_E \) and \( X_I \) are the edge and interior framing joint forces respectively. The elastic stiffness parameter \( k_e \) is a measure of...
the relative stiffness of the nails in the direction perpendicular to the sheathing edge to the flexural stiffness of the framing stud member. Interior framing stud members of multi-panel shear walls (Fig. 2.11) are connected to two nail lines, and the nails from each slip in the same direction across the framing stud member. Consequently the effective elastic stiffness parameter $K_{e,I}$ of an interior framing stud member is $K_{e,I} = \frac{1}{2} K_{e,E}$, where $K_{e,E}$ is the elastic stiffness parameter of the edge framing stud member. The wall load $V$ of multi-panel walls is therefore,

$$V = \frac{\beta B}{c} p_{j,\text{max}} \tag{2.47}$$

where

$$\beta = \frac{n + \frac{2}{3} \left( \phi_{x,E} + \phi_{x,I} (n - 1) \right)}{(1 + r_s^2)^{\frac{1}{2}}} \tag{2.48}$$

$$r_s = \frac{6 + 2 \left( \phi_{x,E} + \phi_{x,I} \right)}{7} \tag{2.49}$$

and $\phi_{x,E}$ and $\phi_{x,I}$ are the framing joint force reduction factors corresponding to $K_{e,E}$ and $K_{e,I}$ for the edge and interior framing stud members respectively.

Wall load $V$ for multi-panel walls is not strictly proportional to the number of sheathing panels $n$, as seen by Eq. 2.47 to 2.49. However, because framing stud flexibility does not significantly influence wall load $V$ (Table 2.3), the approximation $\phi_{x,I} = \phi_{x,E}$ leads to the simplified equation

$$V = \frac{nBk\Delta_i}{c} \left(1 + \frac{2}{3} \phi_{x,E} \right) \tag{2.50}$$

or in terms of the maximum nail force $P_{j,\text{max}}$
\[ V = \frac{(1 + \frac{2}{3} \phi_{X,E})}{(1 + \frac{2}{3} r_s)} \cdot \frac{nBP_{j,\text{max}}}{c} \]  

(2.51)

Table 2.4 compares the dimensionless parameter \( c/VBP_{j,\text{max}} \) of an \( n \) panel wall given by the original Eq. 2.48 and the simplified Eq. 2.50, and shows the simplified equation overestimates wall load \( V \) by less than 3%. The wall load \( V \) is shown to be essentially proportional to the number of sheathing panels.

<table>
<thead>
<tr>
<th>Number of panels</th>
<th>( \beta = \frac{V_c}{BP_{j,\text{max}}} ) from Eq. 2.47</th>
<th>( n(1 + 2/3 \phi_{X,E}) ) from Eq. 2.50</th>
<th>( V_c/BP_{j,\text{max}} ) from Eq. 2.50</th>
<th>( 2/1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.905</td>
<td>0.905</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.792</td>
<td>1.810</td>
<td>1.01</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>3.538</td>
<td>3.621</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>8.775</td>
<td>9.050</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>17.503</td>
<td>18.100</td>
<td>1.03</td>
<td></td>
</tr>
</tbody>
</table>

\( K_{e,E} = 0.05 \quad \phi_{X,E} = 0.652 \)

\( K_{e,I} = 0.025 \quad \phi_{X,I} = 0.544 \)

As previously noted, each interior framing stud member is connected to two adjoining sheathing panels. These panels slip in opposite directions along the length of the framing stud member such that the framing stud member itself is not axially loaded. When the external wall load \( V \) is uniformly distributed along the length of the wall, the axial force in the top framing plate is small, being a maximum of \( X_I \) at the connection to the interior framing stud member. Table 2.5 summarises the equations for a multi-panel shear wall having sheathing panel aspect ratio \( a = 2 \).
### Table 2.5 Summary of Equations for Multi-Panel Shear Walls having sheathing aspect ratio $\alpha = 2.$

<table>
<thead>
<tr>
<th>Notation</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Design Load</strong></td>
<td>$V = \frac{1 + 2/3\phi_{x,E}B_{p,m}}{(1 + r_{s}^{1/2})/c}$</td>
</tr>
<tr>
<td><strong>Framing Plates</strong></td>
<td></td>
</tr>
<tr>
<td>- Axial force</td>
<td>$P_{p,max} = \frac{1}{3/\phi_{x,E} + 2}$</td>
</tr>
<tr>
<td>- Bending moment</td>
<td>$M_{p,max} = 0.014 \frac{BV}{n}$</td>
</tr>
<tr>
<td>- Shear force</td>
<td>$V_{p,max} = \frac{1}{7n}$</td>
</tr>
<tr>
<td><strong>Edge framing stud</strong></td>
<td></td>
</tr>
<tr>
<td>- Axial force</td>
<td>$P_{sE,max} = \frac{13V}{7n}$</td>
</tr>
<tr>
<td>- Bending Moment</td>
<td>$M_{sE,max} = \frac{0.064\phi_{m,E}}{1 + 2/3\phi_{x,E}} \frac{BV}{n}$</td>
</tr>
<tr>
<td>- Shear force</td>
<td>$V_{sE,max} = \frac{1}{(3/\phi_{x,E} + 2)} \frac{V}{n}$</td>
</tr>
<tr>
<td><strong>Interior framing stud</strong></td>
<td></td>
</tr>
<tr>
<td>- Axial force</td>
<td>$P_{sI,max} = 0$</td>
</tr>
<tr>
<td>- Bending Moment</td>
<td>$M_{sI,max} = \frac{0.128\phi_{m,I}}{1 + 2/3\phi_{x,I}} \frac{BV}{n}$</td>
</tr>
<tr>
<td>- Shear force</td>
<td>$V_{sI,max} = \frac{2}{(3/\phi_{x,I} + 2)} \frac{V}{n}$</td>
</tr>
<tr>
<td><strong>Edge framing joint forces</strong></td>
<td></td>
</tr>
<tr>
<td>- Horizontal force</td>
<td>$X_{E} = \frac{1}{(3/\phi_{x,E} + 2)} \frac{V}{n}$</td>
</tr>
<tr>
<td>- Vertical force</td>
<td>$Y_{E} = V/7n$</td>
</tr>
<tr>
<td><strong>Interior framing joint forces</strong></td>
<td></td>
</tr>
<tr>
<td>- Horizontal force</td>
<td>$X_{I} = \frac{2}{(3/\phi_{x,I} + 2)} \frac{V}{n}$</td>
</tr>
<tr>
<td>- Vertical force</td>
<td>$Y_{I} = 0$</td>
</tr>
</tbody>
</table>

### Nomenclature
- $\phi_{x,E}$ horizontal edge framing joint force reduction factor
- $\phi_{x,I}$ horizontal interior framing joint force reduction factor
- $\phi_{m,E}$ edge bending moment reduction factor
- $\phi_{m,I}$ interior bending moment reduction factor
Fig. 2.12 Construction details of 3.6 m long plywood sheathed shear wall example.

Fig. 2.13 Load-slip relationship for 50 x 2.5 mm diameter nails [34].
2.3.10 *Numerical Example*

Figure 2.12 shows a 3 panel wall, 2.4 m high and 3.6 m long, and sheathed on both faces with 9.0 mm thick plywood. Dry (moisture content less than 18%) green gauged 150 x 50 framing members are used throughout, and are connected to the sheathing by 50 x 2.5 mm dia. galvanised nails at 50 mm centres. The NZS 3603:Table 29 permissible wind/seismic lateral load $P_{WS}$ for 2.5 mm dia. nail of 430N is used in the following example. The corresponding secant nail stiffness $k = 630$ N/mm is given by the load-slip curve for the 50 x 2.5 mm dia. nail shown in Fig. 2.13.

### Table 2.6 Design Actions for 2.4 x 3.6m Long Shear Wall Example.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Proposed model</th>
<th>Walker theory</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nail slip ratio</td>
<td>$r_s$</td>
<td>1.0</td>
</tr>
<tr>
<td>Elastic stiffness parameter - edge</td>
<td>$K_{e,E}$</td>
<td>0.0012</td>
</tr>
<tr>
<td></td>
<td>$K_{e,I}$</td>
<td>0.0006</td>
</tr>
<tr>
<td>Framing joint force reduction factor - edge</td>
<td>$\phi_{x,E}$</td>
<td>0.230</td>
</tr>
<tr>
<td></td>
<td>$\phi_{x,I}$</td>
<td>0.205</td>
</tr>
<tr>
<td>Bending moment reduction factor - edge</td>
<td>$\phi_{m,E}$</td>
<td>0.060</td>
</tr>
<tr>
<td></td>
<td>$\phi_{m,I}$</td>
<td>0.050</td>
</tr>
<tr>
<td>Permissible Wind/Seismic Wall load</td>
<td>$V_{WS}$</td>
<td>51.7 kN</td>
</tr>
<tr>
<td>Framing Plates:</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Axial stress</td>
<td>$f_{t,P}$</td>
</tr>
<tr>
<td></td>
<td>flexural stress</td>
<td>$f_{b,P}$</td>
</tr>
<tr>
<td></td>
<td>shear stress</td>
<td>$f_{s,P}$</td>
</tr>
<tr>
<td>Edge framing studs</td>
<td>Axial stress</td>
<td>$f_{t,E}$</td>
</tr>
<tr>
<td></td>
<td>flexural stress</td>
<td>$f_{b,E}$</td>
</tr>
<tr>
<td></td>
<td>shear stress</td>
<td>$f_{s,E}$</td>
</tr>
<tr>
<td></td>
<td>horizontal joint force</td>
<td>$X_E$</td>
</tr>
<tr>
<td></td>
<td>vertical joint force</td>
<td>$Y_E$</td>
</tr>
<tr>
<td>Interior framing studs</td>
<td>Axial stress</td>
<td>$f_{t,I}$</td>
</tr>
<tr>
<td></td>
<td>flexural stress</td>
<td>$f_{b,I}$</td>
</tr>
<tr>
<td></td>
<td>shear stress</td>
<td>$f_{s,I}$</td>
</tr>
<tr>
<td></td>
<td>horizontal joint force</td>
<td>$X_I$</td>
</tr>
<tr>
<td></td>
<td>Vertical joint force</td>
<td>$Y_I$</td>
</tr>
</tbody>
</table>
Table 2.6 tabulates the elastic stiffness parameters $K_e$, reduction factors $\phi_x$, $\phi_m$, permissible wind/seismic wall load $V_{ws}$, framing stresses $f_t$, $f_b$, $f_s$, and framing joint forces $X$, $Y$ determined from the Table 2.5 equations. The resulting values are also compared with corresponding values given by the Walker theory.

The edge framing stud member resists combined axial and flexural loading, with the maximum flexural stress being about one third the magnitude of the maximum axial stress in this particular case. For an elastic stiffness parameter $K_e = 0.0012$ for the edge framing stud member, the maximum bending moment occurs at a position $y_{t,\text{max}}/H = 0.43$ (see Fig. 2.9) which is about 150 mm from the end of the member. This approximately coincides with the position of maximum axial force. The combined axial and flexural stress at the critical cross-section of the edge framing stud member is greater than the flexural stress at the critical section of the interior framing stud members and framing plate members. The applied shear forces in the framing members are small and may be neglected.

Table 2.7 Influence of Framing Member sizes and Nail Spacing $c$ on Wall Wind-Seismic Load $V_{ws}$ and Elastic Stiffness Parameter $K_e$.

<table>
<thead>
<tr>
<th>Framing member sizes (Nominal)</th>
<th>100 x 50 mm</th>
<th>150 x 50 mm</th>
<th>100 x 100 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheathing Nail spacing $c$ (mm)</td>
<td>$K_e$</td>
<td>$V_{ws}$ (kN)</td>
<td>$K_e$</td>
</tr>
<tr>
<td>150</td>
<td>0.0047</td>
<td>8.7</td>
<td>0.0070</td>
</tr>
<tr>
<td>100</td>
<td>0.0031</td>
<td>13.0</td>
<td>0.0047</td>
</tr>
<tr>
<td>75</td>
<td>0.0023</td>
<td>17.2</td>
<td>0.0035</td>
</tr>
<tr>
<td>50</td>
<td>0.0016</td>
<td>25.6</td>
<td>0.0023</td>
</tr>
<tr>
<td>25</td>
<td>0.0008</td>
<td>50.7</td>
<td>0.0012</td>
</tr>
</tbody>
</table>

The influence of different framing member sizes and nail spacing $c$ on the elastic stiffness parameter $K_e$ and wall wind/seismic load $V_{ws}$ is shown in Table 2.7. The typical range of the elastic
stiffness parameter $K_e$ for common framing member sizes and nail spacing $c$ of shear walls tabulated in Table 2.7 is also shown in Fig. 2.8. Table 2.7 shows an increase in the permissible wind/seismic load $V_{ws}$ of only 5% when the framing size is increased from 100 x 50 mm (nom) to 100 x 100 mm (nom).

Fig. 2.14 Actions on the edge framing stud member plotted against nail spacing $c$ and framing member size. (a) Maximum axial stress $f_{t,e}$. (b) Maximum flexural stress $f_{b,e}$. (c) Horizontal framing joint force $X$. (d) Vertical framing joint force $Y$. 
The influence of framing member size and nail spacing $c$ on the axial force, bending moment, and joint forces of the edge framing stud member is shown in Fig. 2.14. Results from the proposed equations (solid lines) are compared with values determined using the Walker theory for rigid framing stud members, i.e. $\phi_m = \phi_x = 1.0$ (dashed lines). The comparison shows the significant reduction in the flexural stress, and in the horizontal joint force $X$ when the actual flexural stiffness of the framing stud member is allowed for. A striking feature of Fig. 2.14 is that the flexural stress determined from the proposed equation, for a specified nail spacing $c$ is similar for all three framing member sizes despite the different section modulii.

![Diagram](image)

**Fig. 2.15** Load-lozenging deflection response of the 3.6m long plywood sheathed shear wall.
Figure 2.15 plots the wall load $V$ against lozenging deflection $\Delta_y$ (Fig. 2.3) of the multi-panel shear wall shown in Fig. 2.12. Wall lozenging deflection was calculated from the Table 2.1 deflection expression, where the average nail slip $\Delta_{av}$ was given by Eq. 2.45 and the effective secant nail stiffness $k$ was obtained from the load-slip curve shown in Fig. 2.13. The lozenging load-deflection curve shown in Fig. 2.15 is compared with the load-deflection curves corresponding to framing stud members having infinite flexural stiffness, and zero flexural stiffness. Shear wall load-deflection response is observed to be insensitive to framing stud flexibility, confirming the findings by Gupta and Kuo [21].

2.4 ULTIMATE STRENGTH SHEAR WALL MODEL

2.4.1 General

This section describes an ultimate strength model for shear walls. Equations for predicting wall strength $V'$ and the framing joint forces and framing stresses at the ultimate strength of the wall are derived in a similar manner to the elastic design equations described earlier.

In the ultimate strength model, sheathing nail slip is assumed to be sufficiently large for the strength $P'$ of all the nails to be developed. The framing members are assumed to remain elastic. This is a reasonable assumption for shear walls since it is usual for the framing members to remain lightly stressed even after the sheathing nails have yielded. The direction of the relative slip between sheathing and framing (Fig. 2.16) defines the components $P'_x$, $P'_y$ of the nail force $P'$ acting on the framing members in the $x$ and $y$ directions.

Strength equations are initially derived for the single panel wall of height $H$ and width $B$ shown in Fig. 2.16(a). Only walls having a sheathing aspect ratio $\alpha = 2$ are considered here.
Fig. 2.16 Ultimate strength of shear wall.  (a) Wall displacements.  (b) Nail force distribution.  (c) Force distribution in framing members.
Fig. 2.17 Nail slip diagram. (a) Localised nail slip and nail force components. (b) Nail load-slip relationship and definition of nail secant stiffness $k_s$.

Fig. 2.18 Force distribution, deflected shape, moment diagram, shear force diagram and axial force diagram at wall ultimate strength $V'$ for the right hand framing stud member.
2.4.2 Framing Behaviour

2.4.2.1 Framing Member Joint Forces

Figure 2.18 shows the free body diagram of the right hand, i.e. tension framing stud member, and the corresponding shear force, bending moment and axial force diagrams. Horizontal joint forces \(X\)' act at the ends of the framing stud member to maintain equilibrium with the nail force components acting in the direction perpendicular to the framing stud member.

As in the elastic shear wall analysis described in the previous section, the framing stud member deflects when subjected to sheathing nail forces acting in the direction perpendicular to the framing. The deflected shape of the framing stud member \(\Delta_f\) (Fig. 2.17) is given by Eq. 2.1. The horizontal and vertical slip components of nail \(j\) are the relative movements between the edge of the sheathing and the bent framing stud member, denoted by \(\Delta_{x,j}\) and \(\Delta_{y,j}\) respectively (Fig. 2.17). If all the nails attain strength \(P_j' = P'\), the component of the force of nail \(j\) in the \(x\) direction \(P_{x,j}'\), with reference to Fig. 2.17, is

\[
P_{x,j}' = \frac{\Delta_{x,j} P'}{\sqrt{\Delta_{x,j}^2 + \Delta_{y,j}^2}}
\]

From similar triangles of the nail slip diagram of Fig. 2.17 we have

\[
\Delta_{y,j} = \Delta_x' \quad \quad \quad \Delta_{x,j} = \frac{2\Delta_x' y_j}{H} - \Delta_f
\]

\[
= \frac{2\Delta_x' y_j}{H} - \sum_{i=2,4,6} \infty \sin \left( \frac{i \pi y_j}{H} \right)
\]

where \(\Delta_x'\) and \(\Delta_y'\) represent the relative slip between the corner of the sheathing, and the corner of the framing, (see
Fig. 2.16(a)). The constants $C_i$ were derived previously for the elastic shear wall model using energy methods, and are given by Eq. 2.8).

\[
C_i = \frac{\frac{i}{2} + 1}{4 \Delta_x (-1) i \pi (K_i i^4 + 1)} \quad (2.55)
\]

where $K_i = \frac{cEi_n}{H^4} k_s$ is the inelastic stiffness parameter, and is the ratio of the flexural stiffness of the framing stud member to the secant stiffness $k_s$ of the yielding nails. The inelastic stiffness parameter is the same as the elastic stiffness parameter (Eq. 2.9) except that the secant nail stiffness $k_s$ is used rather than the elastic nail stiffness $k$. The secant nail stiffness $k_s$ is defined with reference to Fig. 2.17(b) as

\[
k_s = \frac{p'}{\Delta'} \quad (2.56)
\]

where $\Delta'$ corresponds to the nail slip at which the ultimate nail force $p$ occurs.

Substituting Eqs. 2.53 and 2.54 into Eq. 2.52 gives

\[
P_{x,j}' = \frac{2y_j \sum_{i=2,4,6}^{\infty} C_i \sin \left( \frac{i \pi y_j}{H} \right)}{p' \left( \frac{2y_j \sum_{i=2,4,6}^{\infty} C_i \sin \left( \frac{i \pi y_j}{H} \right)}{H} + \frac{r_s^2}{r_s^2} \right)} \quad (2.57)
\]

where $r_s = \frac{\Delta'_y}{\Delta'_x}$ is the nail slip ratio of the panel. The magnitude of $r_s$ required in the solution of Eq. 2.57 is derived later.

Moment equilibrium of the forces acting on the framing stud member about the origin $O_s$ (Fig. 2.16 (c)) and solving for the horizontal framing joint force $X'$ gives
\[ x' = \frac{4p'}{Hc} \int_{2y}^{51} \left( \frac{2y - \sum_{i=2,4,6...}^{\infty} C_i \sin \left( \frac{i\pi y}{H} \right)}{H} \right)^2 dy \] (2.58)

Integrating Eq. 2.58 gives

\[ x' = \phi_x' \frac{P'B}{x} \] (2.59)

where \( \phi_x' \) is the horizontal framing joint strength parameter, and is plotted in Fig. 2.19 and 2.20 against \( r_s \) and for framing stud stiffness parameter \( K_1 \).

The vertical framing joint force \( Y' \) can be determined in a similar manner to the horizontal joint force \( X' \). Moment equilibrium of the forces acting on the rigid framing plate member about the origin \( O_p \) (see Fig. 2.16(c)) gives

\[ y' = \frac{4p'}{Bc} \int_{0}^{B} \frac{r_s x^2}{\sqrt{(2r_s x/B)^2 + 1}} \] (2.60)

and integrating;

\[ y' = \phi_y' \frac{P'B}{y} \] (2.61)

where \( \phi_y' \) is the vertical framing joint strength parameter, and is plotted in Fig. 2.19 against \( r_s \).
Fig. 2.19 The strength parameters $f_s$, $f_p$, $f_x$, $f_y$ plotted against nail slip ratio $r_s$, $r_{ss}$, $r_{sp}$ for values of the inelastic stiffness parameter $K_I$. 
Fig. 2.20 The framing joint force strength parameter $\phi^x$ plotted against nail slip ratio $r_s$ and the inelastic stiffness parameter $K_I$.

2.4.2.2 Framing Member Axial Force

With reference to Fig. 2.18, the maximum axial force $P'_{s,max}$ in the right framing stud member occurs at the bottom, and from overall equilibrium is

$$\frac{P'_{s,max}}{B} = \frac{HV'}{B} - Y' = 2V' - Y'$$

(2.62)

where $V'$ is the ultimate strength of the wall. When the load $V'$ is uniformly distributed along the top framing plate member (see Fig. 2.16(a)) the maximum axial load $P'_{p,max}$ in the framing plate member is equal to the horizontal framing joint force $X'$. 

2.4.2.3 Framing Member Bending Moment

Moments equilibrium about position $y_n$ gives (see Fig. 2.18) the expression for the bending moment $M'_{s}$ in the framing stud member

$$M'_{s} = X' \left( \frac{H}{2} - y_n \right) - \int_{y_n}^{H/2} P'_x(y)(y - y_n) \, dy \quad (2.63)$$

where $P'_x(y)$ is the component of the nail force per unit length acting in the $x$ direction and is

$$P'_x(y) = \frac{P'}{c} \frac{\sum_{i=2,4,6,\ldots} \frac{2y}{H} \sin \left( \frac{i\pi y}{H} \right)}{\sqrt{\left[ \sum_{i=2,4,6,\ldots} \frac{2y}{H} \sin \left( \frac{i\pi y}{H} \right) \right]^2 + r_s^2}} \quad (2.64)$$

Integrating Eq. 2.63, the maximum bending moment $M'_{s,\text{max}}$ is

$$M'_{s,\text{max}} = \frac{13.85 \times 10^{-3} \phi_{m,E} H^2 P'}{c} \quad (2.65)$$

and $\phi_{m,E} = M'_{s,\text{max}} / M'_{s,\text{max}} (K_I = \infty, r_s = 1.0)$ is the bending moment strength parameter for the edge framing stud member, and is plotted in Fig. 2.21 against nail slip ratio $r_s$ and inelastic stiffness parameter $K_I$. The distribution of bending moment (Eq. 2.63) along the top half of the framing stud member is plotted in Fig. 2.22 in terms of the maximum bending moment $M'_{s,\text{max}}$ (Eq. 2.65), and for $K_I = \infty$, 0.01 and 0.001.

Similarly, the maximum bending moment $M'_{p,\text{max}}$ in the framing plate member is

$$M'_{p,\text{max}} = \frac{13.85 \times 10^{-3} \phi'_{m,p} B^2 P'}{c} \quad (2.66)$$
and $\phi_{m_p}$ is the bending moment strength parameter for the framing plate member, and is plotted in Fig. 2.21 against nail slip ratio $r_s$.

![Graph showing bending moment strength parameter](image)

**Fig. 2.21** The bending moment strength parameter $\phi_m$ plotted against nail slip ratio $r_s$ and the inelastic stiffness parameter $K_I$. (a) For $K_I = \infty$. (b) For finite values of $K_I$. 
Fig. 2.22 Framing stud bending moment $M'_s$ plotted against framing stud height for values of inelastic stiffness parameter $K_I$.

2.4.2.4 Framing Member Shear Force

With reference to Fig. 2.18 the maximum shear force $V'_s,\text{max}$ in the framing stud member occurs at the ends of the member, and is equal to the horizontal framing joint force $X'$ (Eq. 2.59). Similarly, the maximum shear force $V'_i,\text{max}$ in the framing plate member is equal to the vertical framing joint force $Y'$ (Eq. 2.61).
2.4.3 **Nail Slip Ratio** $r_s$

The $y$ component $P'_{y,j}$ of the force $P'_j$ of nail $j$ located along the framing stud member (Fig. 2.17) is

$$P'_{y,j} = \frac{P' \Delta y_{j}}{\sqrt{\Delta x_{j}^2 + \Delta y_{j}^2}}$$  \hspace{1cm} (2.67)

Substituting Eqs. 2.53 and 2.54 into Eq. 2.67, the sum of vertical nail forces acting along the stud framing member $BC$ (Fig. 2.16) is

$$\sum_{BC} P'_{y,j} = \frac{2r_s P'}{c} \int_{0}^{H/2} \frac{1}{\sqrt{\frac{2y}{H} - \sum_{i=2,4,6...} C_i \sin \left( \frac{i\pi y}{H} \right)}^2 + r_s^2} \ dy$$  \hspace{1cm} (2.68)

Integrating Eq. 2.68 gives,

$$\sum_{BC} P'_{y,j} = \frac{\phi'_s P' H}{c}$$  \hspace{1cm} (2.69)

Where $\phi'_s$ is the framing stud strength parameter and is plotted in Fig. 2.19 against $r_s$ for values of inelastic stiffness parameter $K_I = \infty$, 0.01 and 0.001.

Similarly the sum of nail force components $P'_{x,j}$ acting in the $x$ direction along the framing plate members $\overline{AB}$ is

$$\sum_{\overline{AB}} P'_{x,j} = \frac{2P'}{c} \int_{0}^{B/2} \frac{1}{\sqrt{1 + \left(2r_s x/B\right)^2}} \ dx$$  \hspace{1cm} (2.70)

Integrating Eq. 2.70 gives

$$\sum_{\overline{AB}} P'_{x,j} = \frac{\phi'_s P' B}{c}$$  \hspace{1cm} (2.71)
where $\phi'_p = \sinh^{-1}(r_s)/r_s$ is the framing plate strength parameter, and is plotted in Fig. 2.19 against nail slip ratio $r_s$.

Moment equilibrium of all the nail forces $P$ about the origin $O$ (see Fig. 2.16) requires

$$\sum_{j=1}^{N} P'_{y,j} x_j + \sum_{j=1}^{N} P'_{y,j} y_j = 0 \quad (2.72)$$

where $N$ is the total number of nails around the perimeter of the sheathing. With respect to framing members $\overline{AB}$, $\overline{BC}$, $\overline{CD}$ and $\overline{DA}$, (Fig. 2.16) Eq. 2.72 can be written

$$\sum_{\overline{AB},\overline{CD}} P'_{y,j} \frac{x_j}{2} - \sum_{\overline{BC},\overline{DA}} P'_{y,j} \frac{y_j}{2} + \frac{4x'_{\overline{H}}}{2} - \frac{4y'_{\overline{B}}}{2} = 0 \quad (2.73)$$

Substituting Eqs. 2.59, 2.61, 2.69 and 2.71 into Eq. 2.73 gives,

$$2\phi'_p \frac{P'B^2}{c} - 2\phi'_s \frac{P'B^2}{c} + 4\phi'_x \frac{P'B^2}{c} - 2\phi'_y \frac{P'B^2}{c} = 0 \quad (2.74)$$

$$\therefore \quad \phi'_p - \phi'_s + 2\phi'_x - \phi'_y = 0 \quad (2.75)$$

The strength parameters $\phi'_p$, $\phi'_s$, $\phi'_x$ and $\phi'_y$ are a function of the nail slip ratio $r_s$ (see Fig. 2.19). There exists one value of $r_s$ that will satisfy the equilibrium requirements of Eq. 2.75, and this can be found through trial and error.
2.4.4 **Shear Wall Ultimate Strength V'**

Horizontal equilibrium of the forces acting along the top framing plate member (Fig. 2.16(c)) requires

\[
v' = \sum_{x,j} p'_{x,j} + 2X'
\]

(2.76)

substituting Eqs. 2.59 and 2.71 into Eq. 2.76 gives,

\[
v' = \frac{P'B}{c} \left( \phi' + 2\phi'_{x} \right)
\]

(2.77)

**Table 2.8 Ultimate strength V' of single Panel Shear Wall.**

<table>
<thead>
<tr>
<th>(K_I)</th>
<th>(r_s)</th>
<th>(\phi'_{p})</th>
<th>(\phi'_{s})</th>
<th>(\phi'_{x})</th>
<th>(\phi'_{y})</th>
<th>(V')</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>1.7</td>
<td>0.768</td>
<td>0.945</td>
<td>0.180</td>
<td>0.180</td>
<td>1.13</td>
</tr>
<tr>
<td>0.01</td>
<td>1.1</td>
<td>0.865</td>
<td>0.960</td>
<td>0.112</td>
<td>0.143</td>
<td>1.09</td>
</tr>
<tr>
<td>0.001</td>
<td>0.8</td>
<td>0.910</td>
<td>0.960</td>
<td>0.075</td>
<td>0.115</td>
<td>1.06</td>
</tr>
</tbody>
</table>

Table 2.8 tabulates the nail slip ratios \(r_s\) satisfying Eq. 2.75; the corresponding strength parameters \(\phi'_{p}\), \(\phi'_{s}\), \(\phi'_{x}\) and \(\phi'_{y}\) and the wall ultimate strength \(V'\) for values of inelastic stiffness parameter \(K_I\). The nail slip ratio \(r_s\) for the particular case of rigid framing stud members is found to be \(r_s = 1.7\) compared to the nail slip ratio \(r_s = 1.4\) given by the elastic model (Eq. 2.34) for rigid framing stud members. As in the elastic model, \(r_s\) decreases with increasing framing stud flexibility (see Table 2.8). Despite the significant reduction in nail slip ratio \(r_s\) with increasing framing stud flexibility, the ultimate strength \(V'\) varies by only 7% in Table 2.8.
2.4.5 *Separation of Framing Members at Corners*

It has been assumed until now that the framing members are pin-jointed at the panel corners. However, tests of shear walls [34] have indicated that separation of the framing joints may occur near ultimate load. In this section, the effect of joint separation on wall strength is examined. Although separation of the framing joints is permitted, it is assumed that the joint can still transfer the framing joint forces $X'$ and $Y'$. This is a reasonable assumption when (as is often the case) nails are used at the connection, since they permit large joint separation while maintaining joint strength. Individual nail slip ratio values for the framing stud member $r_{s,s}$ and framing plate member $r_{s,p}$ are defined, and used to formulate the respective strength parameters $\phi'_s$, $\phi'_x$, $\phi'_p$ and $\phi'_y$ required to satisfy Eq. 2.75. Because joint separation has not been considered until now, the nail slip ratio for each framing member was assumed to be equal to the overall nail slip ratio $r_s$ of the panel $r_{s,s} = r_{s,p} = r_s$. Figure 2.23 shows the separation $\Delta'_{c,x}$, $\Delta'_{c,y}$ of the framing joint in the horizontal and vertical directions. The corresponding nail slip ratio $r_{s,p}$ for the framing plate member, and ratio $r_{s,s}$ for the framing stud member are, with reference to Fig. 2.23 defined as

\[
r_{s,p} = \frac{\Delta'_{y}}{\Delta'_{x}} = \frac{r_s}{1 - r_{c,x}} \tag{2.78}
\]

\[
r_{s,s} = \frac{\Delta'_{y}}{\Delta'_{x} - \Delta_{c,x}} = \frac{r_s}{1 - r_{c,x}} \tag{2.79}
\]

where $r_{c,y} = \Delta_{c,y}/\Delta'$ and $r_{c,x} = \Delta_{c,x}/\Delta'$ are the framing connection slip ratios.
Joint separation leads to an increase in the nail slip ratio $r_{s,s}$ for the framing stud member (see Eq. 2.79), and a decrease in the nail slip ratio $r_{s,p}$ for the framing plate member (see Eq. 2.78). The value of $r_s$ that satisfies Eq. 2.75, is therefore, influenced by joint separation. Table 2.9 tabulates the nail slip ratio $r_s$, the strength parameters $\phi_p$, $\phi_s$, $\phi_x$, $\phi_y$ and the wall ultimate strength $V'$ for particular values of framing connection slip ratios $r_{c,x}$, $r_{c,y}$ and for particular values of the inelastic stiffness parameter $K_I$. Results show that the nail slip ratio $r_s$ satisfying Eq. 2.75 increases with vertical joint separation, and decreases with horizontal joint separation. These results are shown separately in Fig. 2.24, where nail slip ratio $r_s$ is plotted against framing connection slip ratios $r_{c,x}$ and $r_{c,y}$ and the inelastic stiffness parameter $K_I$. The wall ultimate strength is relatively insensitive to separation of the framing connection, varying by only 10% in Table 2.9.
Table 2.9 Ultimate strength $V'$ of Single Panel Shear Wall with Framing Connection Separation.

<table>
<thead>
<tr>
<th>$K_I$</th>
<th>$r_{c,x}$</th>
<th>$r_{c,y}$</th>
<th>$r_s$</th>
<th>$r_{a,p}$</th>
<th>$r_{a,s}$</th>
<th>$\phi'_p$</th>
<th>$\phi'_s$</th>
<th>$\phi'_x$</th>
<th>$\phi'_y$</th>
<th>$V'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>0.0</td>
<td>0.0</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>0.87</td>
<td>0.96</td>
<td>0.11</td>
<td>0.14</td>
<td>1.09</td>
</tr>
<tr>
<td>0.0</td>
<td>0.0</td>
<td>0.5</td>
<td>1.4</td>
<td>0.9</td>
<td>1.4</td>
<td>0.90</td>
<td>0.98</td>
<td>0.10</td>
<td>0.13</td>
<td>1.10</td>
</tr>
<tr>
<td>0.5</td>
<td>0.5</td>
<td>1.1</td>
<td>0.6</td>
<td>2.2</td>
<td>0.95</td>
<td>0.99</td>
<td>0.07</td>
<td>0.09</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0.0</td>
<td>0.7</td>
<td>0.7</td>
<td>2.0</td>
<td>0.91</td>
<td>0.96</td>
<td>0.08</td>
<td>0.12</td>
<td>1.06</td>
<td></td>
</tr>
<tr>
<td>0.001</td>
<td>0.0</td>
<td>0.0</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.91</td>
<td>0.96</td>
<td>0.08</td>
<td>0.12</td>
<td>1.06</td>
</tr>
<tr>
<td>0.0</td>
<td>0.5</td>
<td>1.3</td>
<td>0.8</td>
<td>1.3</td>
<td>0.93</td>
<td>0.95</td>
<td>0.08</td>
<td>0.11</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0.5</td>
<td>1.0</td>
<td>0.5</td>
<td>2.0</td>
<td>0.96</td>
<td>0.99</td>
<td>0.04</td>
<td>0.08</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0.0</td>
<td>0.6</td>
<td>0.6</td>
<td>1.2</td>
<td>0.95</td>
<td>0.98</td>
<td>0.06</td>
<td>0.09</td>
<td>1.07</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 2.24 Nail Slip ratio $r_a$ plotted against joint slip ratio $r_{c,x}$, $r_{c,y}$ for values of inelastic stiffness parameter $K_I$. 

\[ K_I = \begin{cases} \infty & \text{for } \Delta_{CF} = 0 \\ 0.01 & \text{for } \Delta_{CF} = \Delta_{y,x} \\ 0.001 & \text{for } \Delta_{CF} = \Delta_{x,y} \end{cases} \]
Fig. 2.25 External and internal actions in a multi-panel wall at wall ultimate strength $V'$. 

2.4.6 **Strength Equations for Multi-Panel Walls**

The derivation of the strength equations for multi-panel walls is similar to that of the elastic equations for multi-panel walls (see Section 2.3.9). Equilibrium of the top framing plate member of the multi-panel wall shown in Fig. 2.25 requires,

$$
V' = \sum_{x,j} P'_{x,j} + 2X'_{E} + \sum_{k=1,2...} 2X'_{I_k}
$$

(2.80)

where $X'_{E}$ and $X'_{I_k}$ are the edge and interior framing joint forces respectively and $n$ is number of sheathing panels.

Substituting Eqs. 2.59 and 2.71 into Eq. 2.80

$$
V' = \frac{P'B}{c} (n\phi'_{x,E} + 2\phi'_{x,E} + 2(n-1)\phi'_{x,I})
$$

(2.81)

where $\phi'_{x,E}$ and $\phi'_{x,I}$ are the framing joint strength parameters for the edge and interior framing joints respectively. As in the elastic model the wall strength $V'$ is insensitive to framing stud.
flexibility (see Table 2.8), and the strength equation can be simplified by making the approximation \( \Phi_x I = \Phi_x E \) giving

\[
y' = \frac{NB'P'}{c} (\Phi' P + 2\Phi' x, E)
\]

(2.82)

The strength equations predicting framing stresses and joint forces for multi-panel walls are summarised in Table 2.10. Figure 2.26 shows a flow chart summarising the procedure required to solve the Table 2.10 strength equations.

---

**Fig. 2.26** Flow chart for solving strength equations for multi-panel walls.
### Table 2.10 Strength Equations for Multi-Panel Shear Wall having Sheathing Aspect Ratio $\gamma = 2$.

<table>
<thead>
<tr>
<th>Action</th>
<th>Notation</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Ultimate Strength</td>
<td>$V'$</td>
<td>$\frac{nBP'}{c} \phi_p' \left( \phi_p' + 2\phi_x' \right)$</td>
</tr>
<tr>
<td>Anchorage forces</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- horizontal</td>
<td>$V'$</td>
<td>$\frac{HV'}{B}$</td>
</tr>
<tr>
<td>- vertical</td>
<td>$R$</td>
<td></td>
</tr>
<tr>
<td>Framing Plates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Axial force</td>
<td>$P'_p$</td>
<td>$\frac{2P'_B}{X,I} c$</td>
</tr>
<tr>
<td>- Bending moment</td>
<td>$M'_p$</td>
<td>$13.9 \times 10^{-3} \phi'_m,p \frac{P'_B^2}{c}$</td>
</tr>
<tr>
<td>- Shear force</td>
<td>$V'_p$</td>
<td>$\frac{p'_B}{y,E} c$</td>
</tr>
<tr>
<td>Edge framing studs</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Axial force</td>
<td>$P'<em>s</em>{E,\text{max}}$</td>
<td>$(2.3 - \phi'<em>s</em>{E,\text{max}}) \frac{P'_B}{c}$</td>
</tr>
<tr>
<td>- Bending moment</td>
<td>$M'<em>s</em>{E,\text{max}}$</td>
<td>$13.9 \times 10^{-3} \phi'_m,e \frac{P'_B^2}{c}$</td>
</tr>
<tr>
<td>- Shear force</td>
<td>$V'<em>s</em>{E,\text{max}}$</td>
<td>$\frac{p'_B}{x,E} c$</td>
</tr>
<tr>
<td>Internal framing studs</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Axial force</td>
<td>$P'<em>s</em>{I,\text{max}}$</td>
<td>$0$</td>
</tr>
<tr>
<td>- Bending Moment</td>
<td>$M'<em>s</em>{I,\text{max}}$</td>
<td>$27.8 \times 10^{-3} \phi'_m,l \frac{P'_B^2}{c}$</td>
</tr>
<tr>
<td>- Shear force</td>
<td>$V'<em>s</em>{I,\text{max}}$</td>
<td>$\frac{p'_B}{X,I} c$</td>
</tr>
<tr>
<td>Edge framing joints</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- horizontal</td>
<td>$X'_E$</td>
<td>$\frac{P'_B}{x,E} c$</td>
</tr>
<tr>
<td>- vertical</td>
<td>$Y'_E$</td>
<td>$\frac{p'_B}{y,E} c$</td>
</tr>
<tr>
<td>Internal framing joints</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- horizontal</td>
<td>$X'_I$</td>
<td>$\frac{2P'_B}{X,I} c$</td>
</tr>
<tr>
<td>- vertical</td>
<td>$Y'_I$</td>
<td>$0$</td>
</tr>
<tr>
<td>Sheathing Stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f'_s$</td>
<td>$\frac{V'}{nBt'_s}$</td>
<td></td>
</tr>
</tbody>
</table>

**Nomenclature**

- $\phi'_E$ = horizontal edge framing joint strength parameter
- $\phi'_I$ = horizontal internal framing joint strength parameter
- $\phi'_E$ = vertical edge strength parameter
- $\phi'_m,E$ = edge bending moment strength parameter
- $\phi'_m,I$ = interior bending moment strength parameter
2.4.7 **Comparison with Elastic Theory**

In this section, the strength equations are compared with the corresponding elastic theory equations. For the purpose of the comparison, framing members are assumed to be rigid and pin-jointed (i.e. no joint separation), and the maximum sheathing nail force $P_{j,max}$ in the elastic theory equations is assumed equal to the nail ultimate strength $P'$. Table 2.11 compares the equations. The strength theory gives values for wall strength, framing bending moment, and framing joint forces which are on average about 20% greater than those of the elastic theory. The strength theory predicts much greater bending moments in the framing plate members, but predicts values for the framing stud members which are similar to those of the elastic theory. This is attributed to the greater nail slip ratio $r_s = \Delta'_{y}/\Delta'_{x} = 1.7$ of the ultimate strength theory compared to $r_s = 1.4$ for the elastic theory. Thus, the inclination of the nail force $P'$ with respect to the length of the framing plate member is greater in the strength model than in the elastic model, giving larger nail force components acting in the direction perpendicular to the framing plate member.

**Table 2.11 Comparison of Elastic and Strength Theories.**

<table>
<thead>
<tr>
<th>Notation</th>
<th>Elastic theory</th>
<th>Strength theory</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nail slip ratio</td>
<td>$r_s$</td>
<td>$1.43$</td>
</tr>
<tr>
<td>Wall strength</td>
<td>$V'$</td>
<td>$0.955 P'B$</td>
</tr>
<tr>
<td>Framing Plates</td>
<td>$P'$</td>
<td>$0.191 P'B$</td>
</tr>
<tr>
<td></td>
<td>$M'$</td>
<td>$0.013 P'B^2$</td>
</tr>
<tr>
<td></td>
<td>$V'$</td>
<td>$0.137 P'B$</td>
</tr>
<tr>
<td>Framing studs</td>
<td>$P'$</td>
<td>$1.77 P'B$</td>
</tr>
<tr>
<td></td>
<td>$M'$</td>
<td>$0.036 P'B^2$</td>
</tr>
<tr>
<td></td>
<td>$V'$</td>
<td>$0.191 P'B$</td>
</tr>
<tr>
<td>Framing joint force</td>
<td>$X'$</td>
<td>$0.191 P'B$</td>
</tr>
<tr>
<td></td>
<td>$Y'$</td>
<td>$0.137 P'B$</td>
</tr>
</tbody>
</table>
2.4.8 **Numerical Example**

In this section, the Table 2.10 strength equations are used to predict the strength \( V' \) and the associated framing member stresses and framing joint forces of the 3 panel shear wall, shown in Fig. 2.11. The ultimate strength \( P' = 1220 \text{ N} \) of the 50 x 2.5 mm diameter galvanised nails has been taken from test results [34]. This is approximately 2.8 times the permissible NZS 3603:Table 29 wind/seismic nail load \( p_{ws} \) used in the elastic analysis of the wall, see Section 2.3.10. Framing connection slip values \( r_{c',x} = r_{c',y} = 0.2 \) are assumed to represent the joint separation observed in a actual shear wall test, described in Chapter 5.

**Table 2.12 Strength Design Actions for 2.4 x 3.6 m Long Shear Wall Example.**

<table>
<thead>
<tr>
<th>Notation</th>
<th>Strength Model</th>
<th>Ratio with Elastic theory</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nail strength</td>
<td>( P' )</td>
<td>1.22 kN</td>
</tr>
<tr>
<td>Nail slip ratio</td>
<td>( r_{s} )</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>( r_{s,s} )</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>( r_{s,p} )</td>
<td>0.80</td>
</tr>
<tr>
<td>Inelastic stiffness parameter</td>
<td>( k_{t,e} )</td>
<td>0.0060</td>
</tr>
<tr>
<td></td>
<td>( k_{t,i} )</td>
<td>0.0030</td>
</tr>
<tr>
<td>Framing joint strength parameter</td>
<td>( q_{e} )</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>( q_{t,e} )</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>( q_{t,i} )</td>
<td>0.115</td>
</tr>
<tr>
<td></td>
<td>( q_{v} )</td>
<td>0.143</td>
</tr>
<tr>
<td></td>
<td>( q_{v,e} )</td>
<td>0.114</td>
</tr>
<tr>
<td></td>
<td>( q_{v,i} )</td>
<td>0.843</td>
</tr>
<tr>
<td>Wall strength</td>
<td>( V' )</td>
<td>192 kN</td>
</tr>
<tr>
<td>Framing Plates</td>
<td>( f'_{t,p} )</td>
<td>1.4 MPa</td>
</tr>
<tr>
<td></td>
<td>( f'_{t,b} )</td>
<td>5.5 MPa</td>
</tr>
<tr>
<td></td>
<td>( f'_{b,p} )</td>
<td>0.0 MPa</td>
</tr>
<tr>
<td></td>
<td>( f'_{b,b} )</td>
<td>2.8 MPa</td>
</tr>
<tr>
<td></td>
<td>( f'_{s,e} )</td>
<td>17.8 MPa</td>
</tr>
<tr>
<td></td>
<td>( f'_{s,b} )</td>
<td>10.5 MPa</td>
</tr>
<tr>
<td></td>
<td>( f'_{s,b} )</td>
<td>0.8 MPa</td>
</tr>
<tr>
<td></td>
<td>( k'_{e} )</td>
<td>5.3 kN</td>
</tr>
<tr>
<td></td>
<td>( k'_{e,v} )</td>
<td>6.7 kN</td>
</tr>
<tr>
<td></td>
<td>( k'_{e,b} )</td>
<td>0.0 kN</td>
</tr>
<tr>
<td></td>
<td>( k'_{b,v} )</td>
<td>16.7 MPa</td>
</tr>
<tr>
<td></td>
<td>( k'_{b,b} )</td>
<td>1.4 MPa</td>
</tr>
<tr>
<td></td>
<td>( k'_{b,b} )</td>
<td>9.4 kN</td>
</tr>
<tr>
<td></td>
<td>( k'_{b,b} )</td>
<td>0.0 kN</td>
</tr>
</tbody>
</table>
Table 2.12 tabulates the calculated nail slip ratios \( r_s', r_{s,s}' \), \( r_{s,p}' \), strength parameters \( \phi_p', \phi_s', \phi_x', \phi_y' \), wall strength \( V' \), and framing stresses \( f_t', f_b', f_s' \). Where applicable, the maximum stresses and loads are compared with the corresponding values obtained from the elastic analysis (see Table 2.6).

The calculated nail slip ratio for the strength theory is in this case \( r_s = 1.0 \), and is similar to the nail slip ratio for the elastic theory of \( r_s = 1.0 \). The wall strength \( V' \) is found to be 3.7 times the elastic theory wall load, and this is 30% greater than the corresponding nail strength ratio \( P'P_{ws} = 2.8 \) (i.e. nail strength to nail wind-seismic load). A striking feature of the Table 2.12 comparison is that flexural stresses in the framing stud members predicted by the strength theory are approximately 8 times those predicted by the elastic theory, and this is 2.9 times greater than the nail strength ratio \( P'/P_{ws} = 2.8 \). This significant increase is due to the secant nail stiffness \( k_s \) used in the ultimate strength theory being less than the elastic nail stiffness \( k \), and this results in the inelastic stiffness parameter \( K_I \) being 5 times greater than the corresponding elastic stiffness parameter \( k_e \) (see Table 2.12). The resulting maximum flexural stress in the edge framing member is about half of the maximum axial stress.

2.5 CONCLUSIONS

1. An elastic shear wall model has been developed to predict wall stiffness, framing joint forces, framing stresses, and sheathing stresses.

2. An ultimate strength shear wall model has been developed to predict wall strength, and the framing joint forces, framing stresses, and sheathing stresses occurring at the ultimate strength of the wall. Framing joint forces and flexural stresses for the framing stud members are shown to be significant.
3. The proposed models verify that wall strength and stiffness is primarily governed by the sheathing nail load-slip characteristics, and their spacing. The bending of the framing stud members and separation of the framing joints influence the framing joint forces and framing flexural stresses, but have little influence on the stiffness and strength of the wall.

4. The strength of multi-panel walls is shown to be approximately proportional to wall length.
3

SHEATHING NAIL BEHAVIOUR

3.1 INTRODUCTION

The experimental load-slip response of nailed joints has been widely reported [26,27,28]. Many analytical models and empirical relationships have been advanced to describe their load-slip characteristics [29,30,31]. In contrast, there is little information available on the reverse cyclic behaviour of nailed joints. Test programs at M.W.D. Central Laboratories [32,33,34] and at the University of Auckland [35] have provided the only information to date. A description of nailed joint behaviour and a summary of previous investigations is given in Section 3.2.

An experimental study of nailed sheathing to framing joints subjected to reverse cyclic loads, is reported in Section 3.3. The experimental study was primarily undertaken to (i) investigate reverse cyclic behaviour, (ii) identify failure modes and (iii) quantify inelastic displacement capacities.

A simple ultimate strength model for nailed joints is developed in Section 3.5, and together with the constitutive material properties, is used to identify the most significant parameters influencing ultimate strength.
3.2 **BACKGROUND**

This section briefly reviews previous investigations into the load-slip behaviour of nailed joints. Factors influencing joint performance and theories predicting their behaviour are described.

### 3.2.1 General Joint Behaviour

The load-slip behaviour of nailed joints is usually obtained from loading solid timber nailed joints incorporating two or four nails [77]. Nailed joints are loaded either parallel or perpendicular to the direction of the timber grain at a slow loading rate such that ultimate load is reached in 5 to 10 minutes.

Figure 3.1(a) presents the typical form of a load-slip curve obtained from a solid timber joint loaded in shear, in the direction parallel to the grain of the timber. The load-slip curve does not exhibit any linear elastic behaviour or distinct yield point (i.e. limit of proportionality). When the nailed joint is loaded in shear, timber fibres adjacent to the nail shank begin to compress and the nail bends elastically. As the load is increased, the nail yields in bending and the timber crushes inelastically (see Fig. 3.1 (b)). The joint load arising from the bending strength of the nail and crushing strength of the timber is referred to here as the nail bearing load $P_b$. At large joint slip magnitudes the nail is bent through a large angle. So that the bent shape of the nail remains compatible with the joint geometry (Fig. 3.1(b)), one or all of the following must occur:

1. the nail shank withdraws from the joint member holding the nail point (see Fig. 3.1(b)),
2. the nail head penetrates the joint member holding the nail head, or
3. the nail yields in tension.
Fig. 3.1  Load-slip curve for a nailed joint.

For timber joints fabricated from solid timber, nail shank withdrawal is the more common mode. Yielding of the nail in
tension is unlikely because of its high tensile strength [88]. As the nail resists withdrawal, nail tension $T_n$ develops. The component $T_n \sin \theta$ (see Fig. 3.1(b)) in the direction parallel to the loading direction contributes to the total joint load $P$, where $\theta$ is the plastic hinge rotation of the nail shank (see Fig. 3.1(b)). The component $T_n \cos \theta$ acts as a clamping force on the joint members, inducing additional joint resistance $\mu T_n \cos \theta$, where $\mu$ is the coefficient of friction between the timber member surfaces.

The proportion of the total joint load $P$ arising from nail tension $T_n$ varies greatly, and is particularly sensitive to the withdrawal resistance of the nail shank. This withdrawal resistance varies with nail type; nail shanks may be either bright or coated (e.g., galvanised) with either smooth or deformed profiles [29]. Furthermore, the withdrawal resistance reduces with the age of the joint and decreases significantly with moisture content changes [60].

The ultimate strength $P'$ of the joint (Fig. 3.1(a); Point B) depends on the nail bearing load $P$, and the joint load arising from nail tension $T_n$. The ultimate strength of the joint is typically reached between 8 and 12 mm.

The descending branch of the load-slip curve shown in Fig. 3.1 decreases gradually to zero load, with load usually being sustained up to a joint slip exceeding twice that at ultimate strength. Complete failure of the joint occurs as the nail shank, or nail head, pulls out of the timber.

For nailed joints loaded soon after assembly, interface friction may also influence the load-slip characteristics of the joint along the initial portion of the curve $O-A$ (Fig. 3.1(a)). Friction is caused from the axial tension induced in the nail during driving, as described by Malhotra and Thomas [40]. The axial tension appears to reduce with time through relaxation of the timber fibres holding the nail shank, and may even disappear if timber shrinkage occurs.
Also shown in Fig. 3.1 is a typical load-slip curve obtained by Mack [28]. The Mack curve exhibits a yield section, this being attributed to an initial gap of 0.7 mm being deliberately introduced between the joint members. The interface gap delays the joint slip at which nail tension and friction contribute to joint load. The interface gap occurs in practice due to shrinkage of the timber after the joint is fabricated.

The Australian [77] and New Zealand [11]¹ Timber Engineering codes specify a single basic design load \( P_B \) for nailed joints determined as the lesser of:

(i) The lower 1.49%ile characteristic ultimate strength, divided by the load duration and safety factor of 4.15, and

(ii) The lower 1.49%ile load at 0.4 mm slip, divided by a safety factor of 1.25.

Similar procedures are used in other codes [26,67].

Classical linear relationships describing joint stiffness have not proved successful due to the nonlinear behaviour of the joint even at small joint slips (see Fig. 3.1), but more elaborate empirical curvilinear relationships advanced by Mack [27], McLain [81] and Potter [30] have been fitted with good accuracy up to joint slips of 3mm.

Table 3.1 Parameters Influencing Joint Behaviour (Ehlebeck [29]).

<table>
<thead>
<tr>
<th>Materials and Properties</th>
<th>Joint Configuration</th>
<th>Loading Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nail Type</td>
<td>Number of nails/Joint</td>
<td>Kind of loading:</td>
</tr>
<tr>
<td>Nail Size</td>
<td>Single, double, multiple shear</td>
<td>Monotonic, cyclic, reverse cyclic</td>
</tr>
<tr>
<td>Nail Shape</td>
<td>Number of nails/Joint</td>
<td>Kind of loading:</td>
</tr>
<tr>
<td>Nail Surface coating</td>
<td>Interface gap</td>
<td>Monotonic, cyclic, reverse cyclic</td>
</tr>
<tr>
<td>Nail Moisture Content</td>
<td>Nail clinching</td>
<td>Monotonic, cyclic, reverse cyclic</td>
</tr>
<tr>
<td>Mechanical Properties:</td>
<td>Number of nails/Joint</td>
<td>Monotonic, cyclic, reverse cyclic</td>
</tr>
<tr>
<td>- Ductility</td>
<td>Pre-drilled holes</td>
<td>Monotonic, cyclic, reverse cyclic</td>
</tr>
<tr>
<td>- Stiffness</td>
<td>End &amp; edge distances</td>
<td>Monotonic, cyclic, reverse cyclic</td>
</tr>
<tr>
<td>- Bending strength</td>
<td>Spacing of nails</td>
<td>Monotonic, cyclic, reverse cyclic</td>
</tr>
<tr>
<td>- Tensile strength</td>
<td>Angle between nails</td>
<td>Monotonic, cyclic, reverse cyclic</td>
</tr>
</tbody>
</table>

¹ NZS3603: Appendix A [11] proposes that the lower 5 percentile loads adjusted to a standard density be used for determining future design loads in New Zealand.
3.2.1.1 Factors Influencing Joint Behaviour

The load-slip behaviour of nailed joints is influenced by a large number of parameters as summarised (after Ehlebeck [29]) in Table 3.1. The most significant factors are discussed here.

(a) **Nail Diameter and Timber Density**

Brock [26] conducted an extensive experimental study of nailed joints and found the joint load $P$ to be most dependent on nail diameter $d_n$ and timber density $\rho$. The empirical equation $P = k d_n^m$ was fitted to his test data, where $k$ and $m$ are constants. Brock found that the power $m$ ranged from 1.4 at small joint slips to 2.0 at ultimate strength, clearly showing the sensitivity of joint load to changes in nail diameter.

Similar predictions of joint load in terms of density or nail diameter have been proposed by others [28,29,67]. Tests were conducted by Collins [61] on the lateral load-slip behaviour of 3.4 mm diameter nailed joints incorporating solid Pinus Radiata timber. Test variables included timber density, and timber moisture content at assembly and at testing. Collins fitted the empirical equation $P = k d_n^b$ to the test data, and found that the coefficient $b$ varied from 0.3 to 1.2, with an overall average of 0.7 at 0.4 mm slip, and 0.5 at ultimate strength.

(b) **Moisture Content**

Tests by Mack [28,83] showed that load at 0.4 mm slip of joints assembled dry (12% moisture content) and tested dry (dry/dry), was on average 30% higher than that of green/green joints. Similar increases were also apparent at ultimate strength. For joints assembled from green timber and allowed to dry to 12% moisture content over a 12 month period (green/dry), the load at 0.4 mm slip was less than the dry/dry joints. This was probably due to shrinkage of the timber increasing the interface gap. However, the ultimate strength of the green/dry joints was 40% higher than that of the dry/dry joints. Although this phenomenon
could not be explained by Mack at the time, it is possible that the significant increase in the observed ultimate strength was due to corrosion of the plain nail shanks over the 12 months following assembly, increasing the withdrawal resistance of the nail shank.

(c) **Grain Direction with Respect to Direction of Loading**

There is conflicting experimental evidence on the effect of grain direction with respect to direction of loading. For example, findings by Brock [26] and Morris [84] indicated that the load-slip behaviour of nailed plywood or solid joints was not significantly affected by the loading direction with respect to the orientation of the plywood, or the grain direction of solid timber. However, findings by Mack [28] and Potter [87] indicated that the joint stiffness and strength of members loaded perpendicular to the grain was greater than those parallel to the grain. Hunt and Bryant [66] reported that joints loaded perpendicular to the grain showed lower stiffness than joints loaded parallel to the grain at joint slips less than 3 mm, but higher stiffness at larger joint slips.

Discrepancies between the reported results may be due to the tendency for some timber species to split when loaded perpendicular to the direction of the grain, particularly for joints incorporating large nail diameters [87].

(d) **Timber Thickness**

Timber thickness has been shown [26,27] to affect the ultimate strength of a joint and to a lesser extent its stiffness. This is largely due to the different nail bending modes, and is discussed in greater detail later.
Fig. 3.2 Three member nailed joint.

(e) **Nails in Double Shear**

In most codes the permissible load for a three member joint with the nail in double shear (see Fig. 3.2) is given as twice the permissible load of a two member joint with nails in single shear [11]. However, some experimental evidence [26,83] indicates that the design load and ultimate strength of double shear joints is less than twice those of single shear joints.

(f) **Pre-drilling**

Nails are preferred to other timber fasteners because they are easily hand or pneumatically driven without any special joint preparation. Consequently, it is not normal practice to pre-drill nail holes, although it is sometimes required for hardwoods or to decrease nail spacings. Mack [28, 83] found that pre-drilling improved the stiffness and to a lesser extent the ultimate
strength of nailed joints, but this depended on the timber moisture content at testing. The improved performance was attributed to reduced splitting of the timber around the pre-drilled nail holes.

![Diagram of multiple nailed joint](image)

**Fig. 3.3** Multiple nailed joint.

(g) **Multiple Nailed Joints**

Early researchers [26] found that the stiffness and ultimate strength of lapped joints loaded in shear (see Fig. 3.3) were proportional to the number of nails within the joint. Investigations by Stoy [85] confirmed these findings for joints containing up to 20 nails. More recent tests, however, have shown that the stiffness and ultimate strength per nail of long multiple-nailed shear joints decreases with the increasing number of nail rows (defined in Fig. 3.3). Extensive studies by Potter [70] for example, showed a reduction of 4% in joint design load
for every nail row above one. The average reduction in load per nail with increasing number of nail rows, was greatest at small joint slips and least at ultimate strength. The number of nail columns (as defined in Fig. 3.3) had little effect. Studies by Granholm [86] and later Lantos [71] showed that axial strain along the joint members resulted in each nail being subjected to a different joint slip. Consequently, a non-uniform load distribution occurred within the nail group, with the nails furthest from the centre of the joint carrying higher loads than the interior nails. The load distribution was dependent on the axial stiffness of the timber members and the shear stiffness of each nail within the joint, with the reduction in load per nail being greatest for larger diameter nails connecting thin joint members at large spacings. As the ultimate strength is approached, nail load is less sensitive to changes in joint slip, and the load distribution will tend to be more uniform.

(h) Creep

When nailed timber joints are subjected to sustained shear loading, there is a gradual increase in slip with time. The slip may attain a value of several times the instantaneous slip, with the actual magnitude depending on the joint configuration, load level, and load duration, as discussed by Mack [82]. The rate of creep decreases with time, with the majority of creep occurring within the first year [82]. Joint strength is not usually affected by creep.

(i) Loading Rate

Edwards [35] conducted an experimental study of the reverse cyclic behaviour of joints incorporating plywood or steel plate, nailed to solid timber. Results showed that joints subjected to sinusoidal reverse cycle loading applied at a frequency of 1.5 Hz exhibited up to 20% increase in load resistance during the initial load cycle to each amplitude when compared to joints subjected to quasi-static (1.25 mm/min) loading. No apparent
increase in load resistance occurred during repeated dynamic load cycling to the same joint slip, indicating a rate of loading effect on the bearing strength of the timber adjacent to the nail shank. During repeated cycles to the same joint slip there is minimal further crushing of the timber adjacent to the nail. The rate of loading effect was greatest for plywood sheathed nailed joints loaded parallel to the timber grain. The effect was less for steel sheathed joints, and when the joints were loaded in a direction perpendicular to the grain.

Wilkinson [72] subjected solid timber and plywood sheathed joints to harmonic vibration tests. Test apparatus restrictions limited joint loads to low levels and joint slips were not greater than 0.06 mm. Within this range the stiffness of the joints was 1.2 to 2.6 times the stiffness of those loaded statically. However, nailed joints can be expected to sustain large joint slips up to and exceeding their ultimate strength (Fig. 3.1) during a major earthquake, and it would be imprudent to apply the results of these tests to the earthquake behaviour of nailed joints.

Fig. 3.4 Details of the nailed sheathing joint (Walford and Cooper [65]).
3.2.1.2. Nailed Sheathing Joint Behaviour

Walford and Cooper [65] and Gardner [68] tested over 30 different types of joints incorporating wood based sheathing nailed to solid timber. Figure 3.4 shows the joint design chosen to represent the nailed sheathing connection in shear wall and diaphragms. The joints were subject to monotonic quasi-static loading applied in a direction parallel to the edges of the sheathing (Fig. 3.4). Results showed a strong correlation between nail diameter, and joint load occurring at 1.5 mm slip [69].

The ultimate strength of joints occurred at joint slips between 8 and 10 mm for joints incorporating sheathing 9 mm thick or thicker, but occurred at joint slips as low as 4 mm for joints incorporating sheathing only 6 mm thick. Three failure modes were identified:

1. The head of the fastener pulled through the sheathing. This occurred in joints in which the sheathing was 9.0 mm thick or less or in joints with jolt+ head nails.
2. The fastener pulled out of the framing. This mode of failure was confined to flat head nails or staples driven through sheathing 12 mm thick or thicker.
3. The sheathing fractured. This occurred in some of the 6 mm thick sheathing.

An experimental study of the reverse cyclic behaviour of nailed sheathing to framing joints was carried out at M.W.D. Central Laboratories [32,33,34] using test specimens similar to those shown in Fig. 3.4. Figure 3.5 reproduces the load-slip hysteresis loops for a nailed 9 mm thick plywood to framing joint with 50 x 2.5 mm dia. galvanised nails. The hysteresis loops are typical of all joints tested, and are characterised by "pinching"; i.e. loss of stiffness at small joint slips. These tests indicated that the parent curve, (i.e. the envelope of hysteresis loops) was almost identical to the curve obtained from identical test joints subjected to monotonic load.

+ A type of fillet head nail where the nail head is only marginally larger in diameter than the nail shank.
Edwards [35] concluded from his reverse cyclic tests that while joints sheathed with steel plate had higher initial stiffness than plywood sheathed joints, they were significantly less ductile, being prone to low cycle nail fatigue at joint slips of as little as 3 mm.

Fig. 3.5 Load-slip hysteresis loops for a nailed sheathing joint. (Thurston [34]).

3.2.2 Theories Predicting Joint Behaviour

In this section, theories are reviewed which use experimentally determined constitutive properties of the joint materials to predict joint load or joint load-slip behaviour. The theories are reviewed first, and this is followed by a review of material properties reported in the literature.

3.2.2.1 Joint Yield Load

A theory predicting the yield load $P_y$ (see Fig. 3.1) of timber joints incorporating steel dowel fasteners was advanced in 1941 by Johansen [36]. Moeller [37], and later Larsen [38] applied this theory to nailed joints. The theory assumes that the nail
fastener has zero axial stiffness, and both the timber and nail behave in a rigid-plastic manner. Possible modes of failure for a two piece nailed sheathing to framing joint are shown in Fig. 3.6(a). The yield loads for failure modes illustrated in Fig. 3.6(a) are:

\[ P_y = \frac{t_x d f'_r}{1 + \lambda} \left( \lambda + 2\lambda^2 + 2\alpha_c \lambda^2 + 2\alpha_c \lambda^2 + \alpha_c \lambda^3 \right) - \lambda(1 + \lambda) \]  
(3.1)

MODE II \[ P_y = \lambda t_s b,1 \]  
(3.2)

MODE III \[ P_y = a_n b,1 \left[ \frac{2\gamma \lambda}{3(1 + \lambda)} \right]^\frac{1}{2} \]  
(3.3)

MODE IV \[ P_y = t_s n b,1 \left[ \frac{\lambda}{2\lambda + 1} \right] \left( \frac{2(1 + \lambda) + \frac{2\gamma d_n^2(2\lambda + 1)}{3\lambda t_s^2}}{1 - 0} \right)^\frac{1}{2} \]  
(3.4)

where \[ \alpha_c = t_s/t_x \]
\[ \lambda = f'_{b,2}/f'_b,1 \]
\[ \gamma = f'_{y}/f'_b,1 \]

\[ f'_{b,1} \text{ and } f'_{b,2} = \text{the bearing strength of the framing and sheathing respectively} \]

\[ f'_y = \text{yield stress of the nail} \]
\[ t_x = \text{thickness of framing} \]
\[ t_s = \text{thickness of sheathing} \]
\[ d_n = \text{nail diameter} \]

The minimum load determined from Eqs. 3.1 to 3.4 gives the actual yield load \( P_y \). Figure 3.6(b) shows the validity range for failure modes II to IV.
Fig. 3.6 Possible failure modes for a two member nailed joint (Larsen [38]). (a) Deformed shape of nail shank. (b) Validity range.
3.2.2.2 Joint Ultimate Strength

Meyer [39] extended the yield load theory to include the effects of the axial stiffness of the nail. As previously noted, a tensile force $T_n$ develops within the nail shank at larger joint slips. The components $T_n \sin \theta$ and $\mu \sin \theta$ contribute to the shear resistance of the joint, where $\theta$ is the plastic hinge rotation of the nail, as shown in Fig. 3.1(b), and $\mu$ is the coefficient of friction between the contact surfaces of the timber members. The ultimate strength of the joint is the sum of the yield load $P_y$ and the load due to the axial force $T_n$, giving

$$P' = P_y + T_n (\sin \theta + \mu \cos \theta)$$  \hspace{1cm} (3.5)

Mack [28] found good agreement between the experimental ultimate strength of solid timber joints and that predicted from Eq. 3.5

3.2.2.3 Joint Load-Slip Behaviour

Kuenzi [41] applied Hetenyi's [42] beam on elastic foundation theory to predict the load-slip behaviour of nailed joints within their elastic range. Wilkinson [43] produced simplified equations for Kuenzi's theory, and later [44] generalised the theory to deal with joints incorporating dissimilar members. Stluka [45] found that the Kuenzi theory worked well when the nails were driven into smooth pre-drilled holes. Wilkinson [44] found that the generalised theory predicted the stiffness of joints without pre-drilled holes, only after the joint had been precycled. Ehlbeck [29] suggests that the poor agreement between the theory and joints without pre-drilled holes, is due to the timber fibres surrounding the nail distorting during nail driving so that the supporting medium is not homogenous and elastic as assumed.

Analytical nonlinear load-slip models have been advanced by Jansson [46], Foschi [47] and Smith [48]. Jansson proposed a closed form analytical model describing the bearing load-slip behaviour of single shear joints with similar solid timber
members. The model neglects the additional joint load arising from nail tension. Good agreement between Jansson's theory and observed behaviour was obtained. Foschi advanced a nonlinear finite element model for the prediction of the bearing load-slip behaviour of single shear joints incorporating dissimilar members. As in Jansson's model, effects due to nail tension were neglected. Foschi's model was later extended by Malholtra and Thomas [40] to include the effects of interface gaps, initial nail tension due to nail driving, and friction. Smith [48] described a beam on foundation model to predict bolted or nailed joint behaviour. The model accounts for nonlinear material behaviour for the timber and the fastener, tensile stiffness of the fastener, and friction between contact surfaces. Good agreement between theory and experimental results was obtained up to joint slips of 4 mm, as seen in Fig. 3.7.

![Graph](image)

**Fig. 3.7** Theoretical prediction of load-slip response of nailed joint (Smith [48]).
Fig. 3.8 Bearing stress-embedment characteristics of timber.

3.2.2.4 Timber Bearing Stress-Embedment Characteristics

The bearing properties of the timber fibres surrounding the nail strongly influences the behaviour of nailed joints. It is customary to measure the bearing properties of the timber by the method illustrated in Fig. 3.8. The load to "embed" the dowel into the timber is divided by the dowel length and diameter to obtain the bearing stress $f'_b$ at each dowel "embedment" $\Delta_b$. Figure 3.8 presents a typical bearing stress-embedment curve.

A comprehensive review of the literature concerning the bearing properties of timber has been made by Ehlbeck [29]. Some of the important properties of the bearing stress-embedment curve are summarised here.
Fig. 3.9  Ultimate timber bearing strength plotted against nail diameter, timber moisture content and timber density, (Ehlbeck [29]).
Table 3.2 Parameters Influencing the Timber Bearing Strength.

<table>
<thead>
<tr>
<th>Timber</th>
<th>Nail</th>
</tr>
</thead>
<tbody>
<tr>
<td>Species</td>
<td>Diameter</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>Coating</td>
</tr>
<tr>
<td>Density</td>
<td>Shape</td>
</tr>
<tr>
<td>Loading direction to timber</td>
<td></td>
</tr>
<tr>
<td>grain</td>
<td></td>
</tr>
</tbody>
</table>

Ehlbeck [29] summarised the findings of a number of researchers [28, 38, 52, 53, 54, 55] and showed that the bearing stress-embedment characteristics depend on the variables listed in Table 3.2. The majority of research into timber bearing stress properties has been concerned with the bearing strength $f'_b$ of the timber, and very little has been reported about the full bearing stress-embedment curve. Summarising the results of previous experimental studies, Fig. 3.9 shows that the ultimate bearing strength $f'_b$ increases with reducing nail diameter, decreasing moisture content, and increasing density.

![Diagram](a)

![Diagram](b)

**Fig. 3.10** Test methods for determining the bearing stress-embedment characteristics of timber.
The bearing stress-embedment characteristics have been determined by two different test methods. Simes et. al. [53] and Larsen [38] proposed the method illustrated in Fig. 3.10(a) to determine the bearing strength of timber. Posschi [47,52] used a similar method to obtain the full bearing stress-embedment curve. Ehlbeck [29] however, noted that this method is unsuitable for obtaining the bearing stress at small embeddings since the timber fibres do not pass continuously around the dowel. Figure 3.10(b) shows the test method adopted by Jansson [46] and Norén [55] for determining the full bearing stress-embedment characteristics of the timber. The nails were driven into the timber specimen causing the timber fibres to be stretched around the nail shank, thus simulating the actual nailed joint condition. In order to reduce nail bending to a minimum, the timber specimen thickness was not more than four times the nail diameter, and the nail ends were clamped into moment resisting steel brackets. Similar methods were employed by Smith [48] except that the nails or steel dowels were placed in pre-drilled holes.

Marten [56] reported that pre-drilling nail holes increased the bearing resistance of the timber when the nail loaded the timber in a direction parallel to the grain. The increase was not evident when the nail loaded the timber in a direction perpendicular to the grain. Pre-drilling the nail hole resulted in an undisturbed bearing area adjacent to the nail shank. Driving the nail into the timber resulted in local splitting of the timber along the grain. This reduced the effective bearing area adjacent to the nail shank when the nail was loaded in a direction parallel to the grain.

Smith observed that the bearing resistance per unit length of a nail embedded into a timber specimen decreased with increased specimen thickness. The specimen thickness effect was only evident for small nail diameters loading the timber parallel to the grain, and for nail embedment less than 1 mm. Smith concluded that the specimen thickness effect was primarily caused by the bending of the nail shank within the thicker timbers specimens.
Fig. 3.11 Load-withdrawal curve for a nail shank in solid timber.

3.2.2.5 Nail Shank Withdrawal Resistance

Figure 3.11 shows a typical load-withdrawal curve for a nail shank withdrawn from timber. The curve comprises of an initial elastic portion together with a gradually descending branch. The nail shank may be withdrawn about 1 mm before the maximum withdrawal resistance is reached. This deflection is more than just elastic tensile strains in the nail and may include distortion of the timber fibres themselves as they resist the friction forces. Static friction may be higher than sliding friction and the load-withdrawal curve can peak, as shown by dashed line in Fig. 3.11. Once sliding of the nail shank along the timber fibres commences, the withdrawal resistance decreases as the penetration length itself decreases.

Ehlbeck [29] reported that the withdrawal resistance $T_w$ of a nail shank could be represented by the relationship

$$T_w = f A_n; \quad 8d_n < L_e < 20d_n$$

(3.6)

Where $f = N_{r'st}$ = withdrawal resistance factor
where $\mu_{st}$ = coefficient of friction between timber and nail shank,
\[ N_r = \text{radial pressure on nail shank}, \]
\[ A_n = l_e d_n, \text{the contact area between nail and timber}, \]
and $l_e$ = the penetration length of the nail shank having
diameter $d_n$.

Values of $\mu_{st}$ and $N_r$ must be determined experimentally. The
coefficient of friction $\mu_{st}$ varies between nails having different
nail surface coatings (e.g. bright or galvanised) and different
shank profiles (e.g. plain or threaded). When a nail is driven
into the timber the nail point tends to cut some fibres while
other fibres are parted and stretched around the nail shank. The
radial pressure will depend on:
1. The extent to which the timber fibres are cut by the nail
point.
2. The timber density and species.
3. Relaxation of the pressure exerted by the stretched fibres
due to age and seasoning.

For example, Fig. 3.12 shows the significant influence of timber
density on the withdrawal resistance of galvanised and plain nail
shanks, as reported by Collins [61]. His tests also showed that
the withdrawal resistance of plain shank nails in dry timber and
tested some time after assembly was 20% less than when tested
immediately after assembly. Tests by Lawniczack [60] indicated
that the withdrawal resistance of plain nails in green/dry joints
was only 50% of the resistance of dry/dry joints. Lawniczack
[60] also reported that the withdrawal resistance of plain shank
nails varied with cyclic changes in moisture content, as shown in
Fig. 3.13. Galvanised and threaded nail shanks have greater
withdrawal resistance than plain nail shanks, and are not as
sensitive to relaxation and seasoning effects [29,61].
Fig. 3.12 Nail withdrawal resistance as a function of timber density.

Withdrawal Resistance Factor $f$ (MPa)

<table>
<thead>
<tr>
<th>Nail</th>
<th>Plain Shank (length)</th>
<th>Relative Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho$</td>
<td>$f_0$</td>
<td>$f_1$</td>
</tr>
<tr>
<td>Basic Density ($\text{kg/m}^3$)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>630</td>
<td>1.80</td>
<td>1.20</td>
</tr>
<tr>
<td>600</td>
<td>2.00</td>
<td>1.40</td>
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<tr>
<td>500</td>
<td>2.80</td>
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<tr>
<td>400</td>
<td>3.80</td>
<td>2.80</td>
</tr>
<tr>
<td>300</td>
<td>5.00</td>
<td>4.00</td>
</tr>
</tbody>
</table>

Regression $f = a \rho^b$ (MPa)
Fig. 3.13 Nail withdrawal resistance as a function of timber moisture content.

3.2.2.6 Coefficient of Friction

Some guidance is given in the literature on values for the coefficient of friction at timber to timber interfaces. Gober [54] reported coefficient of friction values between 0.3 and 0.5 for roughly sawn contact surfaces of dry European Spruce. Moehler and Maier [62] found that the coefficient of friction increased with decreasing applied normal stress and with increasing moisture content, and that friction was independent of grain orientation. Smith [63] conducted coefficient of friction tests on surfaces between steel and dry European Whitewood and found static and sliding coefficient of friction values of 0.72 and 0.65 respectively.
Fig. 3.14 Stress-strain curve for nail wire.

3.2.2.7 Nail Stress-Strain Behaviour

It is impractical to conduct tensile strength tests on the nails because the finished nail is generally too short to be gripped in tensile testing machines. The parent wire from which the nails are made must be tested instead. Figure 3.14 shows the typical shape of the tensile stress-strain curve for nail wire. The curve is characteristic of cold drawn wire in that it does not exhibit a distinct limit of proportionality or yield plateau.

The ultimate tensile strength of nail wire is typically between 700 and 1100 MPa. The ultimate tensile strength of wire manufactured in New Zealand is tabulated in Table 3.3 [64]. A strong correlation exists between wire diameter and ultimate tensile strength, this being attributed to cold working the wire to the respective diameters. A curvilinear regression of the original data summarised in Table 3.3 gave the expression

\[ f_{ut} = 1282d_n^{-0.32} \text{(MPa)} \]  

(3.7)
where $f_{ut}$ is the ultimate tensile strength in MPa and $d_n$ is the nail diameter in mm.

**Table 3.3 Ultimate Tensile Strengths of New Zealand Cold Drawn Nail Wire [64].**

<table>
<thead>
<tr>
<th>Wire Diameter (mm)</th>
<th>Ultimate tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number of tests</td>
</tr>
<tr>
<td>2.5</td>
<td>108</td>
</tr>
<tr>
<td>2.8</td>
<td>83</td>
</tr>
<tr>
<td>3.15</td>
<td>82</td>
</tr>
<tr>
<td>3.55</td>
<td>98</td>
</tr>
</tbody>
</table>

3.3 **EXPERIMENTAL CYCLIC BEHAVIOUR OF NAILED PLYWOOD TO TIMBER JOINTS**

The experimental study described in this section was primarily undertaken to investigate the failure modes and post elastic behaviour of nailed sheathing to timber joints. In earthquake resistant design, a primary aim is to ensure that the shear wall is capable of deflecting in a ductile manner when subjected to several cycles of lateral loading well into the inelastic range.

Special emphasis was therefore given to determine the performance of nailed sheathing joints during reverse cyclic loading. In addition, the results from the tests enabled validation of the ultimate strength model proposed in Section 3.4, and provided reliable estimates of the distribution of joint strength.
Fig. 3.15 Nailed sheathing joint construction details.

3.3.1 Design and Construction of Specimens

3.3.1.1 Design of Test Series

Test joints were designed to represent the nailed sheathing to framing connection in shear walls and diaphragms. Details of the design are shown in Fig. 3.15. The test joints were fabricated by nailing plywood sheathing to both sides of three 150 x 50 mm green gauged Pinus Radiata rails. Eight "gun" nails were driven into each outside rail, and at least 32 nails were driven into the centre rail. This resulted in the shear connection between the centre rail and the sheathing being much stiffer than the connection to the outside rails into which the test nails were driven. Shear load was applied to the nails parallel to the edge of the sheathing by holding the outside rails and cyclically loading the centre rail (see Plate 3.1).
Nails were spaced at 150 mm and were driven 20 mm in from the edge of the sheathing (Fig. 3.15). While this edge distance was greater than the minimum distance of 3 nail diameters recommended in NZS 3615:1981 [73], it was similar to the edge distances along the edge framing members in an actual shear wall.

Table 3.4 Details of Test Joints.

<table>
<thead>
<tr>
<th>TEST SERIES</th>
<th>JOINT TYPE</th>
<th>Plywood Sheathing</th>
<th>Nails</th>
<th>No. of specimens</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>thickness (mm)</td>
<td>orientation*</td>
<td>Dia. (mm)</td>
<td>length (mm)</td>
</tr>
<tr>
<td>A</td>
<td>AJ12</td>
<td>12</td>
<td>parallel</td>
<td>2.89</td>
<td>50</td>
</tr>
<tr>
<td>A</td>
<td>AJ7L</td>
<td>7.5</td>
<td>perpendicular</td>
<td>2.89</td>
<td>50</td>
</tr>
<tr>
<td>B</td>
<td>BJ12</td>
<td>12</td>
<td>parallel</td>
<td>2.89</td>
<td>50</td>
</tr>
<tr>
<td>B</td>
<td>BJ12P</td>
<td>12</td>
<td>parallel</td>
<td>2.89</td>
<td>50</td>
</tr>
<tr>
<td>B</td>
<td>BJ7</td>
<td>7.5</td>
<td>parallel</td>
<td>2.89</td>
<td>50</td>
</tr>
</tbody>
</table>

*Orientation of surface veneers with loading direction.

Two test series were conducted with each test series incorporating three different joint types, as summarised in Table 3.4. Test variables included plywood thickness, plywood orientation with respect to loading direction, and interface friction. Sheathing and nailing details are summarised in Table 3.4.

Joints in Test Series A were dry/dry joints and were tested within one week of assembly. Joint Types AJ12 and AJ7 incorporated 12 mm and 7.5 mm thick plywood respectively. The plywood was orientated so that the surface veneers were parallel to the loading direction. Joint Type AJ7L was, however, fabricated with the plywood surface veneers orientated in the direction perpendicular to the loading direction. Joint Type AJ7L was otherwise identical to Joint Type AJ7.

Test Series B comprised green dry joints. Joint Types BJ12 and BJ7 incorporated 12 mm and 7.5 mm thick plywood respectively with
the surface veneers orientated in the direction parallel to the loading direction. Thus, except for moisture condition, Joint Types AJ12 and AJ7 were identical to Joint Types BJ12 and BJ7. Joint Type BJ12F incorporated 2 layers of 0.1 mm thick Teflon (P.T.F.E.) between the joint members in order to produce a frictionless interface surface. Joint Type BJ12F was otherwise identical to Joint Type BJ12. A sample size of 27 was selected for Joint Type BJ12 to enable the 5%ile, and 95%ile characteristic load to be determined with 75% confidence [74,75].

3.3.1.2 Materials and Construction

Test Series A

The framing rails were fabricated from Nelson grown No. 1 Framing Grade Pinus Radiata having average Nominal Density \(^+\) of 440 kg/m\(^3\) at an average moisture content of 8.3%. Nelson grown Pinus Radiata was selected because it had density and strength properties that were generally similar to Pinus Radiata grown in the central North Island. The majority of Pinus Radiata in New Zealand is supplied from this latter region. The timber rails were randomly cut from the framing members used in the shear wall test specimens (described in Chapter 4).

The 50 x 2.89 mm diameter zinc coated "gun T" nails (supplied by Able Staples Ltd, Christchurch in magazine form) were hand driven within one week of testing. There was no gap left between the sheathing and framing.

Test Series B

The framing rails were cut from Nelson grown No. 1 Framing Grade Pinus Radiata having an average Basic Denisty \(^++\) of 427 kg/m\(^3\). The timber was from the same pack of timber used for the bearing strength test specimens (described in Section 3.4). The framing rails were ranked according to timber density prior to joint assembly, and then allocated to each test joint so that the distribution of density within Joint Type BJ7, BJ12 and BJ12F was similar.

\(^+\) Nominal Density = Oven dry weight/volume at nominated moisture content.

\(^++\) Basic Density = Oven dry weight/green volume.
Plywood sheathing was manufactured by New Zealand Forest Products, and had an average nominal density of 495 kg/m$^3$ at a moisture content of 11%.

The 50 x 2.89 mm diameter zinc coated 'T' nails (identical to those used in Test Series A) were pneumatically driven into the green joint specimens. Care was taken to ensure that the nail heads were driven flush with the plywood surface. Test specimens were then stored in a controlled environment room for approximately two months, and allowed to cure, firstly, to a moisture content of 12%, and then to a target moisture content of 18%. Shrinkage of the framing rails resulted in a gap between the plywood and rails of about 0.5 mm.

Plate 3.1 View of Test Assembly.
3.3.2 Testing Arrangement

Plate 3.1 shows the test specimen positioned in a test frame. The test frame was bolted into an Instron Universal testing machine which applied compression and tension loads to the centre rail. Tensile loads were transferred between the loading head and the lower end of the centre rail by 2-16 mm diameter tie rods. The outside rails were held in the test frame by bearing plates at the rail ends. Out of plane movement of the outside rails was prevented by screw clamps.

The relative slip between the plywood and framing rails was measured with 2, and 4, linear potentiometers during test series A and B respectively. The potentiometers were mounted at the centre of the test nail group, one on each sheet of plywood for Test Series A, and one at each test nail group in Test Series B. Electrical output from the potentiometers was averaged, and connected to an X-Y plotter together with output from the load cell.

![Graph](image)

**Fig. 3.16** Incremental loading diagram.
The incremental reverse cyclic loading sequence is summarised in Fig. 3.16. The test specimens were tested under displacement control at a rate of 5 mm/min. During the repeated loading cycles to each joint slip, the rate was increased to 20 mm/min in order to reduce testing time.

**Fig. 3.17** Typical load-slip hysteresis loops. (a) Joint Type BJ12: 12 mm thick plywood. (b) Joint Type BJ7: 7.5 mm thick plywood.
3.3.3 Presentation of Test Results

3.3.3.1 Typical Load-Slip Hysteresis Loops

Typical load-slip hysteresis loops of Test Joints BJ12 and BJ7 are shown in Fig. 3.17. Load was sustained during reverse cycling up to joint slips of 10 mm in all joints. The load-slip hysteresis loops for joints incorporating 12 mm thick sheathing (shown in Fig. 3.17(a)) indicate improved stiffness and strength properties compared to joints incorporating 7.5 mm thick sheathing (shown in Fig. 3.17(b)). The shape of the hysteresis loops is similar to the "pinched" hysteresis loops reported by Thurston [34]. For each successive cycle, stiffness degrades at small slips, but increases as the joint slip approaches the previous slip amplitude.

![Graph showing Mean Joint Load vs Joint Slip with annotations: +ve loading direction, -ve loading direction, Initial loading direction, 2nd Cycle Peak.]

**Fig. 3.18** Parent load-slip curves for Joint Type BJ12.

Parent load-slip curves were constructed from the envelope of the load-slip hysteresis loops of Joint Type BJ12. Parent curves were constructed from both positive and negative hysteresis loops, and from the hysteresis loops obtained during the first and second cycle to each slip amplitude. The resulting mean
Fig. 3.19 Parent load-slip curves for joints from Test Series A and Test Series B. (a) Effect of plywood thickness. (b) Effect of surface veneer grain orientation. (c) Effect of interface friction.
parent curves are compared on the same axes in Fig. 3.18. A striking feature of the Fig. 3.18 comparison is that joint load sustained during the first positive excursion (initial loading direction) is greater than the load at the same slip during the first negative excursion. Figure 3.18 also shows that the load sustained at a specified slip amplitude is greater during the first excursion than in the second excursion, for both positive and negative loading directions. The trends observed in Fig. 3.18 are typical of all joints tests.

3.3.3.2 Comparison of Joint Behaviour

Figure 3.19 summarises the mean parent load-slip curves of all test joints in Test Series A and B. The parent curves were constructed from the envelope of the positive hysteresis loops.

Plywood Thickness

The effect of 12 mm thick and 7.5 mm thick plywood on joint behaviour is compared in Fig. 3.19(a). The average strength of Joint Type AJ12 was 40% higher than Joint Type AJ7, but the strength of Joint Type BJ12 was very similar to Joint Type BJ7. The increase in plywood thickness tends to increase the nail bearing load $P_b$, but at the same time reduces the penetration length of the nail within the framing, thus reducing the overall withdrawal resistance of the nail shank. At ultimate load these two effects tend to compensate for each other, explaining the results of Test Series B. Theoretical predictions of joint strength discussed in Section 3.6 confirm this explanation.

The difference between the strength of Joint Types AJ7 and AJ12 may be attributed to the timber density. Only selective density specimens of the framing used in Test Series A were taken, and these showed that the average density of the specimens taken from Joint Type AJ12 was 15% higher than the average density of specimens taken from Joint Type AJ7.
Plywood Orientation

Comparison of load-slip response of Joint Type AJ7 and AJ71 is made in Fig. 3.19(b). The surface veneer grain direction of Joint Type AJ7 was parallel to the loading direction, while that of Joint Type AJ71 was perpendicular to the loading direction. No significant difference in the load-slip behaviour was observed with plywood orientation.

Interface Friction

Comparison of the load-slip behaviour of Joint Type BJ12 and BJ12P is made in Fig. 3.19(c). The two test joint types were identical (including similar distribution of timber density and 0.5 mm interface gap), except that Joint Type BJ12P was fabricated with frictionless Teflon between joint members, so as to isolate the effects of interface friction. Figure 3.19(c) indicates that friction did not contribute to the joint resistance during slips less than between 2 and 3 mm. At ultimate strength, friction contributed up to 25% of the total joint strength.

![Graph](image)

Fig. 3.20 5%ile, 50%ile and 95%ile parent characteristic load slip curves for Joint Type BJ12.
3.3.3.3 Statistical Results of Joint Type BJ12

Figure 3.20 shows the 5%ile, 50%ile and 95%ile parent characteristic load-slip curves for Joint Type BJ12. The parent curve of each test was digitized, and joint load at each specified joint slip was ranked in order from largest to smallest. Joint load was then analysed statistically by a nonparametric approach; i.e. no assumptions were made regarding the underlying distribution of the test data. Thus, for the 27 test specimens, the largest load became the 95%ile characteristic load, given with 75% confidence, [75]. In a similar way, the medium and smallest values became the 50%ile and 5%ile load levels respectively.

The 95%ile strength value is approximately 20% larger than the 50%ile strength value, and approximately 2 times greater than the NZS 3603:1981[11] permissible wind/seismic nail load of 560 N for a 2.89 mm diameter nail.

The three parameter Weibull distribution [76] was found to best model the variation in ultimate strength. Nevertheless, the distribution of strength is also well modelled by assuming a normal distribution. The 95%ile strength value assuming a normal distribution, was estimated within 2% of the nonparametric value shown in Fig. 3.20. The normal distribution represented the distribution of strength better than the lognormal distribution which is specified in some codes [77].
Fig. 3.21 Cyclic behaviour of nailed sheathing joints.
3.3.4 Description of Load-Slip Hysteretic Behaviour

In this section the observed load-slip hysteretic behaviour of the test joints is discussed. Fig. 3.21 illustrates the observed cyclic response of nailed joints shown in Fig. 3.17. Both positive and negative load-slip hysteresis loops are superimposed, onto the same axes of Fig. 3.21(a). Curve (+1) (Fig. 3.21(a)) shows the first loading cycle in the positive (initial) loading direction. During this the nail bends and the timber adjacent to the nail crushes inelastically, as discussed in Section 3.2 and shown in Fig. 3.21(b). Axial tension in the nail shank causes the nail point to withdraw from the framing and the nail head to penetrate and crush the sheathing (see Fig. 3.71(b)). Joint resistance at Point A is, therefore, due to the nail bearing load \( P_b \), and the load resulting from nail tension \( T_n \). Nail tension and the corresponding withdrawal of the nail point from the framing is shown in Fig. 3.21(e). When load is released after the excursion to Point A, the nail will unload along a load-withdrawal path having stiffness greater than or equal to the initial elastic portion of the curve, such that at zero load the nail point is partially withdrawn from the timber (see Fig. 3.21(e)).

Loading in the negative direction, i.e. curve (-1) of Fig. 3.21(a) (Fig. 3.21(a): curve (-1)) causes reverse bending of the nail shank and crushing of the timber as illustrated in Fig. 3.21 (c). The load-slip hysteresis loop in the negative direction and positive direction (curves (-1) and (+1)) have similar shape, with the load resistance in the positive excursion being slightly greater than that in the negative excursion. The difference in load resistance is due to nail tension in the negative excursion being less than nail tension in the positive excursion, and this is a result of the previously crushed timber under the nail head and the partially withdrawn nail point.

During a subsequent excursion to the same slip amplitude (Fig. 3.21(a): curves (+2) and (-2)) the nail shank is unsupported
within the cavity of crushed timber (Fig. 3.21(c)), and the applied load is resisted by the cantilever bending strength of the nail within the cavity.

As the previous slip amplitude (Fig. 3.21(a): point D and E) is approached the nail shank becomes supported by the previously crushed timber, increasing joint stiffness and strength. The inclination of the nail induces nail tension, but this is again less than that obtained during the first excursion (see Fig. 3.21(e)). The load-slip hysteresis loops have identical shape in both loading directions.

Curve (+3) represents the first positive excursion to the next amplitude. This load-slip curve follows the previous curve (+2) up to point D showing that there is minimal stiffness degradation after the second excursion to each slip amplitude. After point D, the timber adjacent to the nail shank continues to crush. The increased inclination of the nail shank causes nail tension to develop, resulting in additional nail point withdrawal and crushing of timber under the nail head (Fig. 3.21(e)). The load resistance quickly approaches the previous maximum, and the parent curve is then closely followed. Consequently, there can be extensive cycling within a specified amplitude without affecting joint behaviour at larger slip amplitudes.

The difference between curve (+3) and curve (-3) is due to the effects of reduced nail tension. Load up to the intersection of curve (-3) with the previous unloading curves, arises from the bending strength of the nail within the crushed cavity. Nail tension is low because of the previous crushing of the sheathing, and withdrawal of the nail point during the positive excursion curve (+3). At the intersection itself, the nail becomes fully supported along the crushed timber, and because nail tension is low, the load resistance will be limited to the nail bearing load $P_b$ (see Fig. 3.21(a)).
Fig. 3.22 General load-slip relationship for 12 mm and 7.5 mm thick plywood sheathed joints

3.3.5 Failure Mechanisms

Figure 3.22 illustrates the typical parent load-slip response of test joints incorporating 7.5 mm and 12 mm thick plywood. The 12 mm thick plywood joints exhibited improved strength properties when compared to the 7.5 mm thick plywood joints, having a more stable ductile failure mode.

3.3.5.1 Joint Types AJ7, AJ7L and BJ7: Joints Incorporating 7.5 mm Thick Plywood

All the nails in test joints incorporating 7.5 mm thick plywood were bent in single curvature as shown in Fig. 3.6(a) Failure Mode IV. Joint failure occurred between slips of 7 and 20 mm and was characterised by low cycle nail fatigue, and by the nail heads pulling through the sheathing.
Fig. 3.23 Failure of 7.5 mm thick plywood sheathed joints.

At joint slips greater than about 7 mm, the nail head began to penetrate the surface of the plywood, damaging the surface veneers (as shown in Fig. 3.23(a)). Reverse cycling of the joint led to further damage under the nail head (see Fig. 3.23(b)), and at joint slips of 10 mm some of the nail heads pulled completely through the plywood. Plate 3.2 shows the condition of the plywood around the nail head at the end of the test. Nail fatigue occurred in many of the remaining nails (even after a few cycles to slips less than 10 mm) due to the high reverse curvature demands on the nail shanks. Nail fatigue and nail head
pull-through resulted in a complete loss of load carrying capacity, and this was the reason for the sudden drop in the falling branch of the parent load-slip curve illustrated in Fig. 3.22.

Fig. 3.24 Failure mode of Joint Type BJ12.

3.3.5.2 Joint Types AJ12, BJ12 and BJ12P: Joints Incorporating 12 mm Thick Plywood.

The failure mode of the 12 mm thick plywood joints is shown in Fig. 3.24, and has some features of both Failure Mode III and Failure Mode IV (see Fig. 3.6(a)). Figure 3.24 shows that while some reverse bending of the nail occurred within the plywood, the plywood was of insufficient thickness to enforce full nail head fixity.
In the 12 mm thick plywood joints there was generally less degradation of the plywood surface veneers under the nail head than that observed in the 7.5 mm thick plywood joints. The thicker plywood provided a much improved grip of the nail shank around the head, and nail tension caused the nail point to withdraw from the framing, as shown in Fig. 3.25(a). During reverse cycling, and as the joint returned past the zero slip position, the partially withdrawn nail shank was forced out through the sheathing, as shown in Fig. 3.25(b). When the nail was loaded in reverse bending, the nail shank tended to jamb within the sheathing, and the nail point withdrew further from the framing member, as shown in Fig. 3.25(c). The withdrawal process was repeated during each excursion, progressively pulling the nail shank out of the timber. At the end of the test the nail heads protruded between 10 and 30 mm from the plywood surface.

Plate 3.2 Condition of Plywood at the end of Test.
Fig. 3.25 Failure of 12 mm thick plywood sheathed joints.

In contrast to the failure mode of the nails driven through 7.5 mm thick plywood, very few of the nails in the 12 mm thick plywood joints failed through low cycle fatigue. This can be attributed to the progressive withdrawal of the nail point causing the plastic hinge to form in a new part of the nail shank during each excursion.

Clearly this valuable ductile mechanism would be lost if the clamping action of the sheathing was less than the withdrawal
resistance of the nail shank. The limits on joint geometry that enforce this mechanism are uncertain, and further experimental studies are necessary to define them.

3.4 MATERIAL CHARACTERISTICS

This section presents the results of tests used to determine the material properties required as input data for the ultimate strength model (described in Section 3.5).

3.4.1 Timber Bearing Stress-Embedment Characteristics

3.4.1.1 Test Specimens and Procedure

An experimental study was undertaken to obtain the bearing stress-embedment characteristics of Pinus Radiata. The study comprised of Test Series A and B. The series A specimens were designed to determine the influence of fastener diameter, timber density, moisture content and loading direction, on the bearing properties of Pinus Radiata. Test Series B was included to investigate the influence of loading rate on the timber bearing properties.

![Diagram of Test Specimens](image)

**Fig. 3.26** Details of Test Series A bearing test specimens.
Test Series A:

Six hundred 100 mm square test specimens of 6, 10 and 12 mm thickness were prepared from 50 lengths of 150 x 50 mm Nelson grown No. 1 framing grade Pinus Radiata as illustrated in Fig. 3.26. The Basic Density of the specimens varied between $320 \, \text{kg/m}^3$ and $540 \, \text{kg/m}^3$ while the mean Basic Density was $422 \, \text{kg/m}^3$. Timber moisture content at the time of preparing the specimens was greater than 30%.

Table 3.5 Bearing Test Specimens.

<table>
<thead>
<tr>
<th>TEST</th>
<th>Fastener dia. (mm)</th>
<th>Loading direction with respect to timber grain</th>
<th>Number of specimens</th>
<th>Specimen thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1P</td>
<td>1.88 staple</td>
<td>parallel</td>
<td>100</td>
<td>6</td>
</tr>
<tr>
<td>1N</td>
<td>1.88 staple</td>
<td>perpendicular</td>
<td>100</td>
<td>6</td>
</tr>
<tr>
<td>2P</td>
<td>2.89 nail</td>
<td>parallel</td>
<td>100</td>
<td>10</td>
</tr>
<tr>
<td>2N</td>
<td>2.89 nail</td>
<td>perpendicular</td>
<td>100</td>
<td>10</td>
</tr>
<tr>
<td>3P</td>
<td>3.33 nail</td>
<td>parallel</td>
<td>100</td>
<td>12</td>
</tr>
<tr>
<td>3N</td>
<td>3.33 nail</td>
<td>perpendicular</td>
<td>100</td>
<td>12</td>
</tr>
</tbody>
</table>

The specimens in Series A were grouped into six test categories, as summarised in Table 3.5. Three fastener diameters were tested: 1.8 mm diameter staple, 2.89 mm diameter nail and 3.33 mm diameter nail. For each diameter, two hundred specimens were tested: one hundred with the fastener loading the timber in a direction parallel to the grain (i.e. Table 3.5: Specimens 1P, 2P and 3P), and one hundred with the fastener loading the timber in a direction perpendicular to the grain (i.e. Table 3.5: Specimens 1N, 2N, 3N). The distribution of density within each test category was similar. After the fasteners were pneumatically driven into the specimens, the specimens were stored in a constant environment room for approximately two months and allowed to dry to moisture contents between 12% and 25% (mean of 18%).

Figure 3.27 shows the testing apparatus. A steel frame was clamped around the edges of the specimen to prevent buckling and
splitting. The fastener was clamped into steel side brackets to enforce fixity at the supports, and thus reduce bending. The maximum deflection of the fastener was estimated to be less than 0.01 mm. A 1 mm gap provided between the face of the specimen and the steel bracket permitted crushed timber under the fastener to protrude out from the face of the specimen without applying pressure to the steel brackets. Frictionless Teflon (P.T.F.E.) strips placed above and below the fastener held the specimen vertical. A linear potentiometer mounted at the side of the specimen measured the embedment of the fastener into the timber. The fasteners were forced through the timber at a loading rate of 5 mm/min. Bearing load and fastener embedment were recorded on an X-Y plotter, and then digitised for subsequent data analysis. The moisture content of each specimen was determined immediately after testing.

Fig. 3.27 Testing arrangement for Test Series A bearing test specimens.
Fig. 3.28 Testing arrangement for Test Series B bearing test specimens.

Test Series B:

Three lengths of 150 x 50 Nelson grown Radiata Pine framing were prepared into 30 – 150 mm long test specimens. The average Nominal Density of the timber was 436 kg/m³ at an average moisture content of 9%.

Figure 3.28 illustrates the testing arrangement. Four 2.5 wide by 6.0 mm long hardened steel dowels were manufactured with 2.0 mm long conical points so that the dowels separated the timber fibres as a nail would, when driven into the specimen prior to testing. Two sides of a 3.2 mm diameter dowel were machined to produce the 2.5 mm wide oval shaped dowel, (Fig. 3.28). Deflection at the tip of the dowel was estimated to be less than 0.1 mm.

Each specimen was subjected to two tests. The dowels were firstly embedded through the timber at a (quasi-static) loading rate of 5 mm/min. Secondly, the dowels were driven adjacent to,
but not less than 10 mm from, the previous test holes, and then embedded through the timber at a (dynamic) loading rate of approximately 1500 mm/min. All specimens were tested immediately after the driving of the dowels. Bearing load and dowel embedment were recorded on the CEDACS data acquisition system [99] at intervals of 0.005 seconds.

**Fig. 3.29** 5%ile, 50%ile and 95%ile bearing stress-embedment curves for Test Specimens 2P and 2N.
3.4.1.2 Presentation and Discussion of Test Results

In this section the results from the bearing stress-embedment tests are summarised:

1. Bearing Stress - Embedment Model

Figure 3.29 shows the experimental data points for Test Specimens 2P and 2N (see Table 3.5) at the 5\%ile, 50\%ile and 95\%ile levels. The following procedure was adopted to obtain the experimental points shown -

a. Test specimens having moisture content $\omega$ outside the range $16\% < \omega < 20\%$ were excluded.

b. Numerical values for the bearing stress were determined at embedments of 0.1, 0.2, 0.4, 0.6, 0.8, 1.0, 1.5, 2.0, 3.0, 4.0 and 5.0 mm.

c. The bearing stress at each embedment was ranked in increasing magnitude such that

$$f_{b,1} < \ldots < f_{b,1} < \ldots < f_{b,n}$$

d. The cumulative probability points were determined for a sample size $n$ using the relation [58]

$$F (f_{b,i}) = i/(n + 1) \quad (3.8)$$

e. Individual specimen bearing stresses $f_{b,i}$ were plotted against the cumulative probability points $F (f_{b,i})$, as in Fig. 3.30.

f. The cumulative probabilities at the 5\%ile, 50\%ile and 95\%ile levels were approximated by linear interpolation.

The following exponential expression proposed by Foschi [47] (see Fig. 3.8 for notation),

$$f_b = K_b (\Delta_o + \Psi_b) (1-e^{\Delta_o/\Delta_b}) \quad (3.9)$$
was matched to the test data using the least squares fitting technique [59]. Figure 3.29 shows the fitted exponential curves for Test Specimens 2P and 2N, and shows that the bearing stress-embedment curves are well represented by Eq. 3.9. Table 3.6 tabulates the Eq. 3.9 best fit parameters for each test category in Test Series A.

**Table 3.6 Bearing Stress Parameters.**

<table>
<thead>
<tr>
<th>Dia.</th>
<th>Direction</th>
<th>Exclusion limit</th>
<th>$K_b$ (N/mm$^2$)</th>
<th>$\Delta_0$ (mm)</th>
<th>$\Psi$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.08</td>
<td>parallel</td>
<td>5%</td>
<td>75</td>
<td>0.32</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50%</td>
<td>142</td>
<td>0.22</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95%</td>
<td>317</td>
<td>0.20</td>
<td>1.5</td>
</tr>
<tr>
<td>1.08</td>
<td>perpendicular</td>
<td>5%</td>
<td>73</td>
<td>0.27</td>
<td>7.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50%</td>
<td>151</td>
<td>0.21</td>
<td>6.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95%</td>
<td>302</td>
<td>0.14</td>
<td>4.9</td>
</tr>
<tr>
<td>2.09</td>
<td>parallel</td>
<td>5%</td>
<td>72</td>
<td>0.28</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50%</td>
<td>112</td>
<td>0.23</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95%</td>
<td>225</td>
<td>0.17</td>
<td>0.5</td>
</tr>
<tr>
<td>2.89</td>
<td>perpendicular</td>
<td>5%</td>
<td>75</td>
<td>0.32</td>
<td>7.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50%</td>
<td>112</td>
<td>0.31</td>
<td>8.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95%</td>
<td>141</td>
<td>0.43</td>
<td>8.1</td>
</tr>
<tr>
<td>3.33</td>
<td>parallel</td>
<td>5%</td>
<td>80</td>
<td>0.23</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50%</td>
<td>149</td>
<td>0.19</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95%</td>
<td>224</td>
<td>0.17</td>
<td>0.4</td>
</tr>
<tr>
<td>3.33</td>
<td>perpendicular</td>
<td>5%</td>
<td>101</td>
<td>0.25</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50%</td>
<td>131</td>
<td>0.27</td>
<td>5.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95%</td>
<td>152</td>
<td>0.34</td>
<td>5.9</td>
</tr>
</tbody>
</table>

![Fig. 3.30 Cumulative probability plotted against bearing stress at embedments of 0.1 mm and 5.0 mm for Test Specimen 2P.](image)

Fig. 3.30 Cumulative probability plotted against bearing stress at embedments of 0.1 mm and 5.0 mm for Test Specimen 2P.
(a) Parallel

(b) Perpendicular

Plate 3.3 View of Bearing Test Specimens at Completion of Test.
2. Failure Modes

For specimens 1P, 2P and 3P in which the nail shank was embedded parallel to the timber grain, the nail caused crushing and cleavage of the timber fibres as shown in Plate 3.3(a). When the load was released, the load dropped rapidly with little deformation recovery.

For specimens 1N, 2N and 3N in which the nail shank was embedded perpendicular to the timber grain, the failure mode developed in five stages as embedment increased:

Stage I: Crushing of the timber fibres. This can be considered as analogous to the crushing of hollow straws.

Stage II: Splitting of the timber along the grain (see Plate 3.3(b)).

Stage III: The timber fibres act in tension similarly to stretched ropes loaded transversely.

Stage IV: The stretched fibres fail in tension (see Plate 3.3(b)) leading to a sudden drop in bearing stress, as shown by the bearing stress-embedment curve in Fig. 3.31.

Stage V: Bearing stress continues to increase after each tension failure as more timber fibres are crushed and loaded in tension.

When the load was released, the bearing stress-embedment curve recovered along a path having a lower stiffness than the initial elastic portion of the curve.
Fig. 3.31 Typical bearing stress-embedment curve when the nail loads the timber in a direction perpendicular to the grain.

Fig. 3.32 Scatter diagram plotting bearing strength against timber Basic Density for Test Specimens 2P and 2N.
Table 3.7 Relationship Between Bearing Stress and Nail Diameter, Timber Moisture Content and Timber Density.

<table>
<thead>
<tr>
<th>LOADING DIRECTION</th>
<th>VARIABLE</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
<th>R</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>parallel</td>
<td>$K_b$</td>
<td>0.0663</td>
<td>-0.2274</td>
<td>-0.1913</td>
<td>1.3111</td>
<td>0.55**</td>
<td>41**</td>
</tr>
<tr>
<td></td>
<td>$\Delta_0$</td>
<td>1.4276</td>
<td>0.0448</td>
<td>-0.4544</td>
<td>-0.0308</td>
<td>0.31*</td>
<td>10**</td>
</tr>
<tr>
<td></td>
<td>$\psi$</td>
<td>0.0171</td>
<td>-3.63 x 10^{-3}</td>
<td>-0.135 x 10^{-3}</td>
<td>4.78 x 10^{-6}</td>
<td>0.23</td>
<td>6**</td>
</tr>
<tr>
<td>perpendicular</td>
<td>$K_b$</td>
<td>0.1085</td>
<td>-0.0558</td>
<td>-0.4012</td>
<td>1.3235</td>
<td>0.52**</td>
<td>35**</td>
</tr>
<tr>
<td></td>
<td>$\Delta_0$</td>
<td>0.0273</td>
<td>0.1884</td>
<td>-0.2449</td>
<td>0.4975</td>
<td>0.30*</td>
<td>9**</td>
</tr>
<tr>
<td></td>
<td>$\psi$</td>
<td>0.0940</td>
<td>-7.63 x 10^{-3}</td>
<td>+0.12 x 10^{-3}</td>
<td>4.32 x 10^{-6}</td>
<td>0.13</td>
<td>2</td>
</tr>
</tbody>
</table>

* Statistically significant at 5% level. \(d_n\) (mm)
** Statistically significant at 1% level. \(\omega\) (%)
\(\rho(Kg/m^3)\)

3. The Effect of Timber Density, Timber Moisture Content and Nail Diameter on the Specimen Bearing Stress

Figure 3.32 shows a scatter diagram of timber bearing strength $f'_b$ vs timber density for Test Specimens 2P and 2N, having a moisture content $\omega$ between 16% and 20%. Figure 3.32 shows a moderate correlation between bearing strength and timber density.

A multiple curvilinear regression [59] of all the Test Series A data for both parallel and perpendicular loading directions gave the equations shown in Table 3.7. The dependent variables were the initial stiffness $K_b$, yield embedment $\Delta_0$ and bilinear factor $\psi$ (see Fig. 3.8 for notation). These are the parameters required to define the best fit exponential bearing stress-embedment curve, Eq. 3.9. The independent variables were the timber
moisture content $\omega$, the timber density $\rho$ and the nail diameter $d_n$.

The most significant variable influencing bearing strength is density, followed by moisture content, and least significantly nail diameter.

The hypothesis that the populations of the original test data with different nail diameters have identical means was tested by the nonparametric Mann Whitney Test Statistic [58]. No significant differences (1% level) between the means were observed for the perpendicular loading direction. In other words, it is uncertain whether the small differences between the means (see Fig. 3.33) are due to nail diameter or to variation in the test data. Some significant differences were however, observed for the parallel loading direction.

The diameter effect on the parallel specimens and not the perpendicular specimens is explained as follows: larger nail diameters caused more splitting of the timber along the grain (see Plate 3.3) than smaller nail diameters, irrespective of loading direction. This splitting reduced the nail bearing area for those nails loaded parallel to the grain, but did not affect the bearing area for those loaded perpendicular to the grain.
Fig. 3.33 Bearing stress-embedment curves plotted for values of nail diameter, timber moisture content and timber Basic Density.
Fig. 3.33 (Continued).
Bearing stress-embedding curves from the multiple curvilinear regression (Table 3.7) are shown in Fig. 3.33 for the parallel and perpendicular loading directions. The comparisons highlight the significance of timber density and moisture content. Figure 3.34 compares the bearing stress-embedding curve for 3.0 mm diameter nail loaded through timber parallel and perpendicular to the grain, with density of 400 kg/m³ and moisture content of 15%. Both curves exhibit a similar initial elastic region, but the perpendicular to the grain specimens have a significantly higher bilinear stiffness than the parallel to the grain specimens. This is attributed to the perpendicular to the grain timber fibres being stretched in tension as described earlier in this section.

![Graph showing bearing stress-embedding curves](image)

**Fig. 3.34** Bearing Stress-embedding curves for parallel and perpendicular to the grain specimens.
Fig. 3.35 Bearing stress-embedment curves for quasi-static and dynamic loading.

4. The Effect of Loading Rate on the Specimen Bearing Stress

Figure 3.35 summarises the experimental results of Test Series B undertaken to investigate the effects of loading rate on the bearing stress-embedment curve. When the nail loaded the timber parallel to the grain, a 15% increase in bearing stress was observed due to dynamic loading. The increase is statistically significant.

3.4.2 Effects of Loading Rate on the Nail Withdrawal Resistance in Pinus Radiata

Results from a limited investigation into the effects of loading rate on the withdrawal resistance of plain zinc coated nails are presented in this section. Static and dynamic nail withdrawal tests on 50 x 2.89 mm diameter zinc coated 'T' nails in Pinus Radiata were undertaken. Thirty specimens were prepared having
an average Nominal Density of 420 kg/m$^3$ at a moisture content of 9.8%. Six nails were hand driven 37 mm (excluding nail point length) into the edge of each timber specimen and then immediately withdrawn. Two tests were conducted with each specimen. For the first test the nails were withdrawn at a (quasi-static) rate of 20 mm/min. In the second test, nails were driven adjacent to but not less than, 10 mm from the previous nail positions, and then withdrawn at a (dynamic) loading rate of 1500 mm/min. Load and nail withdrawal were recorded on the CEDACS data acquisition system [99] at intervals of 0.005 seconds.

The average withdrawal resistance (Eq. 3.6) for the dynamic tests was 15% higher than that obtained from the quasi-static tests. The difference is statistically significant.

3.4.3 Coefficient of Friction

Tests were undertaken to establish values for the coefficient of static friction at timber to timber and timber to steel contact surfaces.

![Diagram of testing arrangement](image)

**Fig. 3.36** Testing arrangement for determining coefficient of friction values at contact surfaces between framing and plywood.
3.4.3.1 Coefficient of Friction Values for Contact Surfaces of Pinus Radiata

Figure 3.36 shows the simple testing apparatus employed to determine the coefficient of friction values at interfaces between Pinus Radiata framing and plywood. The apparatus was designed to simulate the frictional forces which had developed in the sheathing nailed joint specimens reported in Section 3.3. The test specimens consisted of two Pinus Radiata framing rails in contact with a sheet of plywood. The framing had a Basic Density of 422 kg/m³ and moisture content of 17%. The plywood surface veneers were parallel with the loading direction.

A force normal to the contact surface, provided by a deadweight, was applied to the plywood through eight 6 mm diameter studs (see Fig. 3.36). Three normal load magnitudes were applied to the specimens, as detailed in Table 3.8. The magnitude of the load applied at each stud was selected as representative of the likely clamping force imposed by nail fasteners. Fifteen specimens were fabricated, and were tested initially with the lowest normal load level, and then with an increased normal load for each subsequent test.

<table>
<thead>
<tr>
<th>NORMAL LOAD/STUD</th>
<th>393 N</th>
<th>611 N</th>
<th>785 N</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVERAGE COEFFICIENT OF FRICTION</td>
<td>0.47</td>
<td>0.40</td>
<td>0.39</td>
</tr>
</tbody>
</table>

Table 3.8 Coefficient of Friction Values Applicable at Contact Surfaces Between Pinus Radiata Framing and Pinus Radiata Plywood.

Average experimental coefficient of friction values for each normal load level are tabulated in Table 3.8. Consistent with observations by Moehler and Maier [62] (Ehlbeck [29]), the coefficient of friction decreased with increasing normal load.
3.4.3.2. Coefficient of Friction Values for Contact Surfaces of Pinus Radiata and Plain Nail Shank

Figure 3.37 shows the testing arrangement for the coefficient of friction tests between plain nails and Pinus Radiata framing. The plain nails were represented by a 2.9 mm diameter bright steel rod. Ten framing specimens were fabricated. The average timber Basic Density was 422 kg/m$^3$ and moisture content was 17%. Normal loads of 200 N, 400 N, 500 N and 600 N were applied to the specimens. The steel rod was embedded into the timber as the normal load was applied. The steel rod was pulled (50 mm/min) through a different location on the framing for each normal load test.

Figure 3.38 shows the least squares best fit linear relationship with a correlation coefficient of 0.91. The resulting average coefficient of friction was 0.60.

![Diagram](image)

**Fig. 3.37** Testing arrangement for determining coefficient of friction values at contact surfaces between framing and nail shank.
3.4.4 Moment Curvature Behaviour of 50 x 2.89 mm diameter 'T' Nails

The moment-curvature response of the 50 x 2.89 mm diameter zinc coated 'T' nails used in the nailed joint tests reported in Section 3.3 is described in this section.

Figure 3.39 shows the testing apparatus. After removing the nail head, the nail shank was clamped into two rigid pin ended swing arms, (see Fig. 3.39). One swing arm was mounted on rollers that permitted free horizontal movement parallel to the axis of the nail shank. Equal horizontal loads on each of the swing arms produced constant bending moment over the 20 mm length of the nail shank.

The relative movement of two small pointers clamped 10 mm apart on the nail shank enabled curvature within this gauge length to be measured. For each load increment, the bent nail and pointers were photographed, the distance between the ends of the pointers measured from an enlarged projection of the negative, and the curvature calculated within the gauge length.
Fig. 3.39 Testing arrangement for determining moment-curvature relationship of nails.

Fig. 3.40 Moment-curvature relationship for 2.89 mm diameter zinc coated 'T' nail.
Figure 3.40 shows the experimental moment curvature points for the 2.89 mm diameter zinc coated nails. The initial elastic portion of the curve is shown to be well predicted by the classical moment-curvature relationship.

3.5 ULTIMATE STRENGTH MODEL

3.5.1 General

A simple ultimate strength model extending the theory advanced by Meyer [39] is presented in this section. In the Meyer model (Eq. 3.5), the joint ultimate strength $P'$ is the sum of the nail bearing load $P_b$ and the additional joint load arising from axial tension in the nail shank.

Various investigators have proposed models for predicting the nail bearing load $P_b$, ranging from the simple yield theory advanced by Moeller [37] and Larsen [38] to the sophisticated nonlinear beam on foundation models [47,48]. The simple yield theory is used in this chapter with minor modifications that allow for the bearing stress-embedment characteristics of the timber. Figure 3.41 shows a nailed sheathing to solid timber joint in which the sheathing thickness is sufficient to enforce double curvature in the nail shank. The loads acting on the nail shank and the resulting bending moment and shear force distribution are also illustrated in Fig. 3.41.

Early researchers assumed rigid-plastic bearing stress-embedment characteristics of timber. In the following theory, the actual bearing stress distribution acting on the nail shank is replaced by an equivalent rectangular stress block distribution incorporating stress block factors $\alpha$ and $\beta$, as shown in Fig. 3.41. The stress block factors themselves are determined by the conditions that the area and the centroidal position of the assumed rectangular stress block distribution, are equal to those of the actual bearing stress distribution (see Fig. 3.41). The stress block factors $\alpha$ and $\beta$ are incorporated into the governing yield theory equations for each failure mode (Fig. 3.6) resulting
Fig. 3.41 Nailed sheathed joint: Failure Mode III.
in equations only a little more complex than the original yield theory equations (Eq. 3.1 to 3.4). The proposed theory results in improved predictions of the relationship between the nail bearing load and joint slip.

The stress block factors $\alpha$ and $\beta$ are derived in Appendix A.

The free body diagram of the nail shown in Fig. 3.41 is based on a number of simplifying assumptions. These assumptions are:

1. The reaction stresses of the supporting timber fibres are related to the embedment of the nail shank at that location, through a specified stress-embedment relationship; i.e. the timber stress at any location is independent of the stress adjacent to that location.

2. The nail penetration into the framing timber (Member 1) is sufficient for a plastic hinge to develop at position $Z$ from the interface.

3. No embedment of the nail shank occurs between the nail head and the plastic hinge located in Member 2, and between the nail point and the plastic hinge located in Member 1. Consequently no bearing stresses act on the nail shank within these regions.

4. Because no bearing stresses act between the nail point and the plastic hinge in Member 2, there is no shear force at position $C$ of the portion of nail shank $C-D$ (see Fig. 3.41). Consequently, shear force in the nail shank at the hinge positions is assumed to be zero.

5. The nail remains straight except at the plastic hinge positions.

It is believed that these assumptions are valid for the large joint slip magnitudes associated with ultimate strength.
Fig. 3.42 Free body diagram of nailed sheathed joint.

Figure 3.42 shows a free body diagram of the nailed joint. The nail bearing load $P_b$ is, with reference to Fig. 3.42, given by

$$ P_b = \frac{N_b}{\cos \theta} \quad (3.10) $$

where $N_b$ is the normal bearing load and $\theta$ is the plastic hinge rotation of the nail shank. An additional nail tension component $T_2$ is also induced, such that the total nail tension $T_n$, is $T_n = T_1 + T_2$. Resolving the nail forces at the joint interface into $x$ and $y$ components gives (see Fig. 3.42)

$$ P_x = T_2 \cos \theta \quad (3.11) $$

$$ P_y = T_2 \sin \theta + P_b \quad (3.12) $$
The force component $P_x$ acts as a clamping force on the joint members inducing friction $\mu^p_{t x}$ parallel to the loading direction, as shown by the free body diagram in Fig. 3.43, where $\mu^p_t$ is the coefficient of friction at the joint interface. Total joint load $P$, arising from the nail bearing load $P_b$ and nail tension, is from the vertical equilibrium of the Fig. 3.43 free body diagram,

$$P = P_y + \mu^p_{t x}$$  \hspace{1cm} (3.13)

$$P = \left(\frac{N_b}{\cos \theta}\right) + T_z(\mu^p_t \cos \theta + \sin \theta)$$  \hspace{1cm} (3.14)

![Free body diagram]

Fig. 3.43 Components of nail forces.

As previously discussed (Section 3.2), at large joint slip magnitudes, the length of the inclined nail shank is greater than its projected length (see Fig. 3.41), and so that the nail remains compatible with the joint geometry, the nail point must withdraw from the framing or the nail head must pull through the sheathing. In this study, it is assumed that the sheathing has sufficient thickness to ensure that the nail point withdraws at ultimate load. The experimental results reported in Section 3.3 showed that nail point withdrawal is preferable to nail head pull-through because it results in more stable ductile joint behaviour. From Fig. 3.41 the nail point withdrawal $\Delta_{n}$ is
\[ \Delta_{\lambda n} = (Z_1 + Z_2) (1 - \cos \theta) \]  
(3.15)

where \( \theta = \sin^{-1} \left( \frac{\Delta_j}{Z_1 + Z_2} \right) \)

and \( Z_1 + Z_2 \) is the distance between plastic hinges. Clearly, the nail point will not withdraw from the framing until \( \Delta_{\lambda n} \) exceeds any initial gap between the joint members.

In the following theoretical predictions, the joint strength \( P' \) has for convenience been defined as the load developed by the joint as the nail point is withdrawn from the framing at a joint slip of 10 mm. This is because experimental evidence [65] indicates that the ultimate strength of typical sheathed joints is developed at slips between 8 and 12 mm. Furthermore, the predicted joint strength has been found to be reasonably insensitive to joint slips between about 8 and 12 mm at which nail withdrawal is assumed to occur.

The ultimate strength of a nailed joint, in terms of the maximum nail point withdrawal resistance \( T_n = T_1 + T_2 \) is from Eq. 3.14

\[ P' = \left( \frac{N_b}{\cos \theta} \right) + \left( \frac{T_n}{N_b \tan \theta} \right) (\mu \cos \theta + \sin \theta) \]  
(3.16)

Theoretical prediction of the normal bearing load \( N_b \) and the nail tension \( T_n \) arising when the nail point withdraws from the framing are discussed separately in the following sections.

3.5.2 Normal Bearing Load \( N_b \)

Derivation of the normal bearing load \( N_b \) is given in Appendix A for joint Failure Modes III and IV (see Fig. 3.6). For Failure Mode III shown in Fig. 3.41, the normal bearing load \( N_b \) is,

\[ N_b = a_1 b_1 b_2 d_n f', \]  
(3.17)

\[ Z_1 = Z_2 \lambda \left( \frac{\alpha_2}{\alpha_1^3} \right) \]  
(3.18)

\[ Z_2 = \left( \frac{4M_p}{P} \right)^{\frac{1}{2}} \frac{\lambda \alpha_2}{\sqrt{f' d_n b_1 n b_2 \lambda (\alpha_1 + 1)}} \]  
(3.19)
\[ M_p = \text{plastic moment of nail shank} \]
\[ \lambda = \frac{f'_{b,1}}{f'_{b,2}} \]
\[ f'_{b,1}, f'_{b,2} = \text{the bearing strength of framing and sheathing respectively in the direction normal to the nail shank}. \]
\[ \alpha_1 = \text{stress block factors in framing member} \]
\[ \alpha_2 = \text{stress block factors in sheathing member} \]

Substituting Eqs. 3.18 and 3.19 into Eq. 3.17 gives

\[
N_b = \sqrt{\frac{4M_p d b_1 n \lambda}{\alpha_2^2 + \frac{\alpha_1 \lambda}{\alpha_2}}}
\]

(3.20)

The normal bearing load \( N_b \) can be determined for a specified joint slip \( \Delta_j \). This is because the stress block factors \( \alpha_1, \beta_1 \) and \( \alpha_2, \beta_2 \) depend on the nail embedment \( \Delta_{bs} \) at the interface (Fig. 3.41) which in turn is related to the joint slip \( \Delta_j \). A closed form solution of Eq. 3.20 does not exist, and a trial and error calculation procedure is required. For a joint slip \( \Delta_j \), calculation begins by selecting trial positions of the plastic hinges \( Z_1 \) and \( Z_2 \). The nail embedments \( \Delta_{bs1} \) and \( \Delta_{bs2} \) at the interface are determined on the assumption that the nail remains straight between plastic hinges (see Fig. 3.41). The nail embedment values \( \Delta_{bs1} \) and \( \Delta_{bs2} \), together with the specified bearing stress-embedment properties enable the stress block factors \( \alpha_1, \beta_1 \) and \( \alpha_2, \beta_2 \) to be calculated (see Appendix A). Finally, the plastic hinge positions \( Z_1 \) and \( Z_2 \) are calculated from Eqs. 3.18 and 3.19. The process is repeated until the determined positions \( Z_1 \) and \( Z_2 \) converge to those assumed at the start of each iteration. A general iterative procedure for finding \( Z_1 \) and \( Z_2 \) is presented in Appendix A. Convergence is rapid (within 2 or 3 iterations) for nailed sheathed joints at slips of 10 mm. Once \( Z_1 \) and \( Z_2 \) are obtained, the normal bearing load \( N_b \) can be determined from Eq. 3.17.
Fig. 3.44 Free body diagram of nail shank in framing member.

3.5.3. **Nail Tension** $T_n$

In this section, the tensile force $T_n$ required to overcome friction and hence cause nail point withdrawal is determined. Figure 3.44 shows a free body diagram of the nail shank within the framing member. The nail shank is assumed to have no bending stiffness; i.e. it is considered as behaving like thin flexible wire. Maximum nail tension $T_n$ occurs just as the nail shank is withdrawn from the timber. Withdrawal resistance is developed by friction forces acting between the nail and timber interface. Withdrawal of the nail shank is resisted by the following three components (see Fig. 3.44):

1. Friction stress $f = \mu \frac{N}{r}$ between the nail shank and the timber between C and D, where $N_r$ is the radial pressure acting on the nail shank (developing at the time the nail
is driven), and \( \mu_{st} \) is the coefficient of static friction at the contact surface between the timber and the nail shank.

The tensile force \( T_r \) in the nail shank at C required to withdraw the embedded nail shank between C and D, is given by the following expression (Ehlbeck [29])

\[
T_r = f \frac{A_n}{n}
\]  \hspace{1cm} (3.21)

where \( f = N_r \mu_{st} \) is the nail withdrawal resistance factor determined by experimental tests on various nail types.

\( A_n = \pi d_n l_e \) = nail surface area on which the radial pressure \( N_r \) acts.

and \( d_n \) = nail diameter

\( l_e \) = effective penetration length of nail shank between the plastic hinge located in Member 1 and the nail point (see Fig. 3.41).

2. Belt friction \( F_p = \mu_{st} N_p \) between B and C, where \( N_p \) is the normal force acting at the bend. \( N_p \) depends on the plastic hinge rotation \( \Theta \) and nail tension \( T_r \) at C. The general belt friction formulation is given elsewhere [101]. The resulting tension \( T_p \) at B is

\[
T_p = T_r e^{\mu_{st}\Theta}
\]  \hspace{1cm} (3.22)

3. Friction \( F_b = \mu_{st} N_b \) between A and B, where \( N_b \) is the normal bearing load due to the nail shank crushing the timber, as shown in Fig. 3.41.
Equilibrium along the nail axis of the Fig. 3.44 free body diagram between A and B requires

\[ T_n = T_b + T_p \]

\[ T_n = \mu_{st} N + T_r e^{\mu_{st} \theta} \]  \hfill (3.23)

**Fig. 3.45** Definition of effective nail penetration length

The effective penetration length \( l_e \) of the nail shank into the framing member prior to nail point withdrawal is given by the Fig. 3.45 geometry as

\[ l_e = l_n - l_p - l_z - t_s \]  \hfill (3.24)

where \( l_n \) is the total nail length, \( l_p \) is the nail point length, \( l_z \) is the length of crushed timber within the framing, and \( t_s \) is the sheathing thickness. In the following analysis, \( l_z \) was taken as 5\( d_n \). Inspection of the nailed joints reported in Section 3.3 showed that \( l_z \) was between 4.5 and 5.0\( d_n \), and this is consistent with findings by others [51].
3.5.4 Predicting the Variability of Joint Strength

The variability of joint strength is assessed by the Monte Carlo simulation technique [76]. The Monte Carlo simulation technique involves repeated calculations of joint strength. For each calculation, random material properties for each material are obtained in accordance with the respective specified probability distribution, from which joint strength is calculated. The results from the large number of joint strength calculations are
treated statistically as if they were experimental results. The flow chart shown in Fig. 3.46 illustrates the simulation technique.

Table 3.9 Variable Properties of Nailed Joint used in Monte Carlo Simulation.

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>NOTATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>WOOD BEARING PROPERTIES</td>
<td></td>
</tr>
<tr>
<td>Parallel to grain</td>
<td></td>
</tr>
<tr>
<td>1. Bearing strength</td>
<td>$f'_{b,P}$</td>
</tr>
<tr>
<td>2. Yield embedment</td>
<td>$\delta_{o,P}$</td>
</tr>
<tr>
<td>3. Bilinear factor</td>
<td>$\psi_P$</td>
</tr>
<tr>
<td>Normal to grain</td>
<td></td>
</tr>
<tr>
<td>4. Bearing strength</td>
<td>$f'_{b,n}$</td>
</tr>
<tr>
<td>5. Yield embedment</td>
<td>$\delta_{o,n}$</td>
</tr>
<tr>
<td>6. Bilinear factor</td>
<td>$\psi_n$</td>
</tr>
<tr>
<td>NAIL PROPERTIES</td>
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<td>7. Ultimate tensile strength</td>
<td>$f_{ut}$</td>
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<td>FRICTION PROPERTIES</td>
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<tr>
<td>8. Nail withdrawal resistance factor</td>
<td>$f$</td>
</tr>
<tr>
<td>9. Friction between joint members</td>
<td>$\mu_c$</td>
</tr>
<tr>
<td>10. Friction between nail and wood</td>
<td>$\mu_{st}$</td>
</tr>
</tbody>
</table>

Table 3.9 tabulates the variable material properties considered in the analysis reported herein. The magnitude and distribution of the bearing stress parameters, and coefficient of friction $\mu_t$ and $\mu_{st}$ were determined from the tests reported in section 3.4. The bearing strength $f'_B$ of the timber in the direction normal to the nail shank (see Fig. 3.41) has been assumed to be related to the experimental bearing strength $f'_d$ of the timber reported in Section 3.4 by the expression

$$f'_B = f'_d \cos \theta$$

where $\theta$ is the orientation of the nail with respect to that in the bearing test specimens (see Fig. 3.47). The bearing stress parameters are considered to be independent of nail diameter for the small range of nail diameters considered in this study. The results of the 2.89 mm diameter bearing test specimens (moisture
content between 16 and 20%, average 18%, and Basic Density between 320 and 540 kg/m³, average 422 kg/m³) forms the data base for the bearing stress parameters.

Fig. 3.47 Direction of nail shank embedment.

The data base for the nail withdrawal resistance factor $f$ (Eq. 3.21) was obtained from tests reported by Collins [61]. These tests involved withdrawing 3.4 mm diameter plain steel and galvanised nails from Pinus Radiata framing (Basic Density between 390 kg/m³ and 513 kg/m³, average 444 kg/m³). Two moisture conditions were included: green/dry and dry/dry. The dry/dry joints were tested after a period of time to permit relaxation of timber fibres holding the nail shank.

The method for generating random values for the timber bearing stress parameters, nail withdrawal resistance factor, and coefficient of friction $\mu_{st}$ and $\mu_{bc}$ is described here. Random variables for each material property were generated using a nonparametric approach; i.e. no assumptions were made regarding the underlying distribution of the variable. The steps followed were:
1. Rank the experimental observations in ascending order such that

\[ x_1 < \ldots < x_i < \ldots < x_n \]

2. Calculate the cumulative probability points for individual tests \( i \) and number of samples \( n \) using the relation [58]

\[ F(x_i) = \frac{i}{(n + 1)} \]

3. Plot the cumulative probability points \( F(x_i) \) against variables \( x_i \) as seen in Fig. 3.48

4. Find random variable \( x \) from the cumulative distribution curve for a value of \( F_x = R_n \), by interpolation between experimental points, where \( R_n \) is a random number between 0 and 1.

![Cumulative Probability Plot](image)

**Fig. 3.48** Cumulative Probability Plot.

Bearing stress parameters for the plywood sheathing (which contained timber veneers orientated in directions parallel and perpendicular to the loading direction) were obtained as follows:
1. Random bearing stress parameters were obtained for each veneer. The bearing stress parameters for each veneer were assumed to be independent.

2. The outside veneer bearing stress parameters were for convenience weighted linearly with respect to their distance from the middle thickness of the plywood. For example, the equivalent bearing strength of the plywood is, with reference to Fig. 3.49 for notation, given by

\[
\sigma'_{b,\text{ply}} = \frac{\sum_{i=1}^{m} \sigma'_{b,i} x_i}{\sum_{i=1}^{m} x_i} \quad (3.25)
\]

This is in recognition that the surface veneers are more highly stressed than the inner veneers.

![Diagram of plywood and veneers]

**Fig. 3.49** Weighting of bearing stress parameters for plywood sheathing.

The plastic moment \( M_p \) of the nail is

\[
M_p = z F_{ut} \quad (3.26)
\]
where \( z_p = \frac{d_n}{6} \) is the plastic section modulus of the nail, 
\( f_{ut} = 1282 d_n^{-0.32} \) (MPa) is the ultimate tensile strength of the nail (see Section 3.2.2), and \( d_n \) is the nail diameter, mm. The ultimate tensile strength \( f_{ut} \) was assumed to have a normal probability distribution, since only limited experimental data was available. The standard deviation \( \sigma_t \) of the ultimate tensile strength \( f_{ut} \) is taken here as \( \sigma_t = 39 \) MPa representing the average value of the data supplied by NZ Wire Industries (Table 3.3). Methods for generating random variables having a normal probability distribution are described elsewhere [78].

Figure 3.50 shows the variation in joint strength at the 5%ile, 50%ile and 95%ile levels with respect to sample size \( n \). Joint strength is shown to be reasonably constant for a sample size \( n \) between 50 and 500. A sample size of 100 was considered adequate for the following parametric study.

![Fig. 3.50 Variation in joint strength with sample size n.](image-url)
3.5.5 Comparison of Proposed Strength Model with Experimental Results

3.5.5.1. Mean Joint Strength

In this section, the results of the ultimate strength model are compared with experimental results of the test joints reported earlier in Section 3.3, and with results reported by Thurston [34] and Walford and Cooper [65]. Joint details including nail type, sheathing thickness, and failure modes, are summarised in Table 3.10. All the test joints reported by Walford and Cooper [65] that incorporated plywood, are included in the table irrespective of whether the joint failed because the nail point pulled out from the framing (denoted P in Table 3.10), or because the nail head pulled through the sheathing (denoted H). Except for Joint Types AJ7 and AJ12 which were tested dry/dry, all joints were tested green/dry. The Basic Density and Moisture Content for each joint is also summarised in Table 3.10, as reported by Thurston [34] and Walford and Cooper [65]. The experimental joint strength tabulated in Table 3.10 is for a single nail, and is assumed to be equal to the reported joint strength, divided by the number of nails within the joint; i.e., load is assumed to be transferred uniformly along the length of the joint. This is a reasonable assumption at ultimate load because the individual nail strength is insensitive to the small changes in nail slip occurring along the length of the joint, as discussed in Section 3.2.1.

The predicted theoretical joint strength given in Table 3.10 was determined using the respective bearing stress parameters, withdrawal resistance factor, and nail tensile strength $f_{ut}$, as described below.

Timber bearing stress parameters and nail withdrawal friction factor were determined from the Table 3.7 and Fig. 3.12 regression equations respectively, for the specified timber density, moisture content, and nail type. For Test Joints AJ7,
Table 3.10 Comparison of Experimental and Theoretical Mean Joint Strength.

<table>
<thead>
<tr>
<th>Test Joint</th>
<th>Nail Type</th>
<th>Nail Coating</th>
<th>Sheathing Thickness</th>
<th>Failure Mode</th>
<th>Basic Density</th>
<th>Moisture Content</th>
<th>Nail Strength $f_{ut}$</th>
<th>Experimental</th>
<th>Theoretical</th>
<th>Differences</th>
<th>Theory Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td>STEWART AJ7 50 x 2.89T</td>
<td>Zinc</td>
<td>7.5</td>
<td>P and H</td>
<td>436</td>
<td>9 (SD)</td>
<td>746</td>
<td>1330C</td>
<td>1583</td>
<td>1.19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>STEWART AJ12 50 x 2.89T</td>
<td>Zinc</td>
<td>12</td>
<td>P</td>
<td>500</td>
<td>9 (SD)</td>
<td>746</td>
<td>1840C</td>
<td>2060</td>
<td>1.12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>STEWART BJ7 50 x 2.89T</td>
<td>Zinc</td>
<td>7.5</td>
<td>P and H</td>
<td>422</td>
<td>18 (GD)</td>
<td>746</td>
<td>860C</td>
<td>1009</td>
<td>1.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td>STEWART BJ12 50 x 2.89T</td>
<td>Zinc</td>
<td>12</td>
<td>P</td>
<td>422</td>
<td>18 (GD)</td>
<td>746</td>
<td>900C</td>
<td>973</td>
<td>1.08</td>
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</tr>
<tr>
<td>THORSTON 30 x 3.15F</td>
<td>galv.</td>
<td>9.0</td>
<td>7</td>
<td>532</td>
<td>16 (GD)</td>
<td>888</td>
<td>1310M</td>
<td>1352</td>
<td>1.03</td>
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<td></td>
</tr>
<tr>
<td>WALFORD 1A 40 x 2.35F</td>
<td>Bright</td>
<td>9.0</td>
<td>P and H</td>
<td>423</td>
<td>18 (GD)</td>
<td>975</td>
<td>1190M</td>
<td>653</td>
<td>0.55</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WALFORD 1B 57 x 1.85</td>
<td>*</td>
<td>9.0</td>
<td>H</td>
<td>423</td>
<td>18 (GD)</td>
<td>1062</td>
<td>1082M</td>
<td>1586</td>
<td>1.47</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WALFORD 2B 57 x 1.85</td>
<td>*</td>
<td>12</td>
<td>P</td>
<td>423</td>
<td>18 (GD)</td>
<td>1062</td>
<td>1507M</td>
<td>1539</td>
<td>1.02</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WALFORD 2C 57 x 2.81T</td>
<td>*</td>
<td>12</td>
<td>P</td>
<td>423</td>
<td>18 (GD)</td>
<td>921</td>
<td>1094M</td>
<td>1134</td>
<td>1.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WALFORD 2D 60 x 2.95J</td>
<td>Bright</td>
<td>12</td>
<td>H</td>
<td>423</td>
<td>18 (GD)</td>
<td>907</td>
<td>1359M</td>
<td>1224</td>
<td>0.90</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WALFORD 2E 75 x 3.15J</td>
<td>Bright</td>
<td>12</td>
<td>7</td>
<td>423</td>
<td>18 (GD)</td>
<td>888</td>
<td>1561M</td>
<td>1602</td>
<td>1.01</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Assumed Bright.

Key: M = Monotonic loading
C = Cyclic loading
P = Nail point pulled out from framing
H = Nail head pulled through sheathing

AJ12, BJ7 and BJ12 the ultimate nail tensile strength $f_{ut}$ was estimated from the test results reported in Section 3.4. For the remaining Test Joints $f_{ut}$ was approximated from the regression equation Eq. 3.7.

Mean coefficient of friction values for timber-timber ($\mu_t = 0.4$), and timber-zinc coated nail shank ($\mu_{st} = 0.6$) were as reported in Section 3.4. Coefficient of friction values for timber-bright nail shank was taken as being equal to that obtained for timber-zinc coated nail shanks. Based on experimental results reported by Collins [61] which showed that the withdrawal resistance of galvanised nail shanks was approximately twice that of bright steel nails, the coefficient of friction value $\mu_{st}$ for timber-galvanised steel (Eq. 3.23) was assumed to be twice the value reported in Section 3.4 for zinc coated steel nails.

Table 3.10 lists the ratio of the predicted strength to the experimental strength and shows the good agreement obtained. The theoretical prediction of joint strength is based on the premise...
that joint failure occurs through the nail point pulling from the framing. When the test joint fails through the nail head pulling through the sheathing (denoted by \( H \) in Table 3.10) the model over-predicts joint strength (e.g. Walford:Test Joint 1B). The cause of the under-prediction of the strength of Walford:Test Joint 1A, however, cannot be explained.

3.5.5.2 5\%ile and 95\%ile Joint Strength Values

In this section, the results from the Monte Carlo simulation are compared with the test results from Test Joint BJ12 reported in Section 3.3. For the comparison the nails were assumed to be manufactured from the same batch of drawn wire. Therefore, any variation in the nail tensile strength \( f_{ut} \) was ignored as being insignificant compared to the variation in other joint material properties.

The distributions of timber bearing properties, and coefficient of friction \( \mu _t \) and \( \mu _{st} \) were extracted from tests reported in Section 3.4. The distribution of nail withdrawal resistance factor \( f \) was as reported by Collins [61].

Table 3.11 Comparison of Experimental and Theoretical 5\%ile, 50\%ile, and 95\%ile Strength Values for Joint Type BJ12.

<table>
<thead>
<tr>
<th>Joint Strength (N) (single nail)</th>
<th>Experimental (N)</th>
<th>Collins [61] withdrawal Friction Factor f</th>
<th>Collins [61] withdrawal Friction Factor f reduced by 20%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Theoretical (N)</td>
<td>Theory Exp.</td>
</tr>
<tr>
<td>5%ile</td>
<td>720</td>
<td>800</td>
<td>1.11</td>
</tr>
<tr>
<td>50%ile</td>
<td>900</td>
<td>1047</td>
<td>1.16</td>
</tr>
<tr>
<td>95%ile</td>
<td>1080</td>
<td>1428</td>
<td>1.32</td>
</tr>
</tbody>
</table>

Table 3.11 compares the experimental and theoretical 5\%ile, 50\%ile and 95\%ile strength values for Test Joint BJ12. There is good agreement between the theoretical prediction of the 5\%ile
and 50\%ile strengths, and the experimental points. The theory however, over-predicts the experimental 95\%ile strength by 32%.

The Monte Carlo simulation predicts slightly higher mean joint strength than that predicted by the theory using mean material properties determined from the regression equations (see Table 3.10). This is because the mean withdrawal friction factor $f$ used for the Monte Carlo simulation was larger than that determined from the Fig. 3.12 regression equation for joint specimen BJ12. The mean density of the nail withdrawal test specimens reported by Collins [61] was larger than that of joint specimens BJ12, as detailed in Section 3.5.3.

The over-estimate of the joint 95\%ile strength is believed to be due to the method by which the withdrawal resistance of the nail shank was determined. The withdrawal resistance of the nail shank may be less than that assumed in the model since:

1. The withdrawal resistance data obtained by Collins [61] was determined from test specimens having timber density variation and moisture conditioning different from that of the joint specimens. Clearly, withdrawal test specimens fabricated from the same nails and timber, and subjected to the same moisture conditioning as the nailed joints would have been preferable in this case.

2. The effective penetration length of the nail shank was less than that assumed in the theory. Figure 3.51 illustrates the deformed shape of the nail within a plywood nailed joint. The model assumed that the nail shank was fixed between the plastic hinge and the nail point. In the test joint, however, some elastic bending of the nail beyond the plastic hinge can be expected, as demonstrated by Poschi and Bonac [52]. This will tend to reduce the withdrawal resistance of the nail in this region, especially when the joint is subjected to reverse cyclic loading.
Fig. 3.51 Effective length of nail shank in framing member.

Table 3.11 also tabulates the theoretical 5\%-ile, 50\%-ile and 95\%-ile strength values for the case where the withdrawal resistance of the nail shank (embedded into the timber between the plastic hinge and the nail point (See Fig. 3.51)), is reduced by 20\%. The tabulated values show that the predicted 95\%-ile strength value is most sensitive and the 5\%-ile and 50\%-ile values less sensitive, to the nail shank withdrawal resistance. Accurate prediction of the 95\%-ile strength value will, therefore, be only possible with improved modelling and improved experimental data for the withdrawal resistance of the nail shank.
3.6 EFFECT OF MATERIAL PARAMETERS ON PREDICTED JOINT STRENGTH

The sensitivity of nailed joint strength to changes in material parameters is investigated in this section. The effects of timber density and moisture contents were determined indirectly from their effects on the bearing stress parameters and withdrawal resistance friction factor. Timber bearing stress parameters and the nail withdrawal friction factor were determined from the Table 3.7 and Fig. 3.12 empirical regression equations for each specified timber density, moisture content and nail diameter.

The effects of nail properties, plywood thickness, and dynamic loading on the joint strength were determined by using the Monte Carlo simulation to predict the 5%ile, 50%ile and 95%ile strength values. Timber bearing stress parameters, the nail withdrawal friction factor, nail tensile strength and coefficient of friction values, were determined by the procedures discussed in Section 3.5.3. For these parameters the effect of timber density variation was reflected through its effect on the bearing stress parameters and nail withdrawal friction factor.

3.6.1 Timber Density

Figure 3.52 shows the variation of mean joint strength (single nail) with nail length \( l_n \) and Basic Density \( \rho_p \) of the timber, for a green/dry joint incorporating a 2.8 mm diameter plain steel (bright) nail driven through 12mm thick plywood. The strength of the joint having Basic Density of 300 kg/m\(^3\) is shown to be about 50% of that for a Basic Density of 500 kg/m\(^3\). The increase in joint strength with increasing density is more significant for longer nails because the withdrawal resistance of the nail point is particularly sensitive to density changes, as detailed by Collins [61] (see Fig. 3.12).
Fig. 3.52 Mean joint strength plotted against nail length and timber density.

Fig. 3.53 Mean joint strength plotted against nail length and timber moisture content.
3.6.2 **Timber Moisture Content**

The general trend in joint strength with variation in timber moisture content is shown in Fig. 3.53. The values plotted are the mean values of a joint incorporating a single 2.8 mm diameter plain steel (bright) nail, 12 mm thick plywood, and timber Basic Density of 422 kg/m$^3$. The variation in green/dry joint strength is shown in Fig. 3.53 with nail length $l_n$ and final moisture content $\omega_f$, and shows that joints of 10% moisture content were approximately 20% stronger than joints with 18% moisture content. Figure 3.53 also shows that the strength of green/dry joints was approximately 20% that of the dry/dry joints. Similar trends were observed for other nail diameters (see Fig. 3.56). This reduction is only applicable to plain steel or zinc coated nails that do not corrode, and further experimental studies are required to determine the effects of the increased withdrawal resistance of rusted plain steel nails on joint strength, as discussed by Ehlbeck [29].

![Graph showing mean joint strength plotted against nail length and nail diameter.](image)

**Fig. 3.54** Mean joint strength plotted against nail length and nail diameter.
Fig. 3.55 Joint strength plotted against nail length and nail diameter.

3.6.3 Nail Properties

The relationship between mean joint strength, nail diameter and nail length is shown in Fig. 3.54 for 12 mm thick plywood sheathing. Values are for timber average Basic Density of 422 kg/m³ and moisture content of 18%. Full results including the 5%ile, 50%ile and 95%ile values are tabulated in Table 3.12. Both nail diameter and nail length are shown to be important parameters influencing joint strength. The ultimate tensile strength $f_{ut}$ of the nail was found to be less significant to joint strength than nail diameter or nail length. A 10% reduction in $f_{ut}$ resulted in less than 2% reduction in joint strength.

Figure 3.55 shows the 5%ile, 50%ile and 95%ile strength values plotted against nail length for a 2.8 mm diameter plain steel (bright) nail driven through 12 mm thick plywood sheathing. The results of Fig. 3.55 show that the sensitivity of joint strength to nail length is greatest at the 95%ile strength value and least at the 5%ile strength value; confirming previous findings that the 95%ile strength value is particularly sensitive to the withdrawal resistance of the nail point.
Figure 3.56 compares the strength of galvanised nails with bright nails for 2.24 mm and 3.15 mm diameter nails driven through 12 mm thick plywood sheathing. Collins [61] reported that the green/dry withdrawal resistance of galvanised nails was about twice that of bright nails. In determining the strength of joints incorporating galvanised nails, the coefficient of friction $\mu_{st}$ was accordingly taken as twice the value determined experimentally for zinc coated nails (see section 3.4). Figure 3.56 shows that the mean strength of joints incorporating galvanised nails is approximately twice the mean strength of joints incorporating bright and zinc coated nails. However, the 95thile strength value of the galvanised nail joints was only 40% greater than that of bright nail joints. This is because the withdrawal resistance of bright or zinc nails has greater variation than that of galvanised nails [61].

Table 3.12 Results from Monte Carlo Simulation.

<table>
<thead>
<tr>
<th>Joint Strength (N)*</th>
<th>Nail diameter $d_n$ (mm)</th>
<th>1.85 mm + staple</th>
<th>2.24</th>
<th>2.50</th>
<th>2.80</th>
<th>3.15</th>
<th>3.55</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5% 50% 95% 5% 50% 95% 5% 50% 95% 5% 50% 95% 5% 50% 95% 5% 50% 95% 5% 50% 95%</td>
<td></td>
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<td>40</td>
<td>802 1020 1364</td>
<td>898 748 959</td>
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</tr>
</tbody>
</table>

* for 12 mm thick plywood
$D_{ave} = 422$ kg/m²
$\omega_{ave} = 18\%$
Fig. 3.56 Mean joint strength as a function of nail length and nail coating.
Fig. 3.57 Mean nail bearing load plotted against nail diameter and plywood thickness

3.6.4 Plywood Thickness

The mean nail bearing load $P_b$ is plotted against nail diameter and sheathing thickness in Fig. 3.57. For nails bending in double curvature, i.e., Failure Mode III, $P_b$ is independent of sheathing thickness and highly dependent on nail diameter. For joints failing in Failure Mode IV, the nail bearing load is dependent on nail diameter and sheathing thickness. A nail diameter of sheathing thickness ratio $d/n/t_s$ of 0.1 to 0.4 is required to enforce Failure Mode III.

The general trend of the relationship between joint strength and plywood thickness is shown in Fig. 3.58. The values plotted are for a 2.8 mm diameter bright nail, with timber Basic Density of 422 kg/m³ and moisture content of 18%. In contrast to the nail bearing load $P_b$ (Failure Mode IV), the overall joint strength is
not significantly influenced by sheathing thickness. This is because the increase in Mode IV nail bearing load \( P_b \) with increasing plywood thickness (Fig. 3.57), is compensated by a reduction in nail tension \( T_n \) associated with the reduced penetration of the nail shank in the framing. The Fig. 3.58 predictions are confirmed by the results of Test Joints BJ7 and BJ12, as reported in Section 3.3. Trends similar to those shown in Fig. 3.58 were predicted at the 95%ile strength value. The 95%ile strength of 9 mm or 16 mm plywood joints was generally within 10% of that of the 12 mm plywood joints.

*Fig. 3.58* Mean joint strength plotted against nail length and plywood thickness.
3.6.5 **Dynamic Loading**

Results from the experimental study reported in Section 3.4 showed that the timber bearing strength, and the withdrawal resistance of zinc coated nails both increased by 15% with dynamic loading. The increase in withdrawal resistance indicates that the coefficient of friction depends on loading rate. In this study, the values of coefficient of friction for timber-timber and timber-nail shank, the nail withdrawal resistance factor, and the timber bearing strength, were all increased by 15%. Results from the Monte Carlo simulation indicate that dynamic loading causes 20% (approx.) increase in the 50\%ile and 95\%ile strength values. The results are in agreement with findings by Edwards [35]. The 20% increase in joint strength is greater than the 15% increase in material properties because of the significant increase in belt friction (see Eq. 3.22), with small changes in the coefficient of friction $\mu_{st}$.

3.7 **Nailed Joint Overstrength $P_o$**

In this section a simple design procedure for establishing the overstrength $P_o$ of nailed joints is presented. The overstrength $P_o$ takes into account all the possible factors that may cause high strength. These include high timber density, low timber moisture content, dynamic load effects, and the effects of joint geometry. The overstrength $P_o$ can be related to the design earthquake strength $P_E$ by

$$P_o = \phi P_E$$  \hspace{1cm} (3.27)

where $P_E = 1.25 P_{ws}$, and $P_{ws}$ is the permissible wind/seismic nail load. The NZS 3603:1981 [11] permissible wind/seismic nail loads were derived from the following empirical relationship [69],

$$P_{ws} = K_1 43.4 d_{kn}^{1.75}$$  \hspace{1cm} (3.28)

where $K_1 = 2.0$ is the load duration factor, and $d_{kn}$ is the nail diameter (mm). Equation 3.28 was found from a curvilinear
regression analysis of joint load at 1.5 mm slip, and nail diameter. The equation is based on test results of green/dry nailed sheathed joints (timber average Basic Density of 423 kg/m$^2$ and moisture content 18%), as described by Walford [69].

The overstrength $P_o$ is given here as the 95%ile strength value determined by the Monte Carlo simulation. A multiple linear regression between the dependent variable $\phi_o = P_o/P_E$ and the independent variables $d_n$ and nail length $l_n$ gives the equation

$$\phi_o = 1.44 - 0.72d_n + 0.06l_n \quad (3.29)$$

where $d_n$ and $l_n$ are in mm. The regression was found from Eq. 3.28, and the 95%ile strength values tabulated in Table 3.12 for bright steel nails driven through 12 mm thick plywood. The Table 3.12 95%ile strength values allows for the variation in material properties, including the effects of timber density. The parametric study reported in Sections 3.6.1 to 3.6.5 also highlighted timber moisture content, loading rate and nail coating (i.e. galvanising) as significant variables influencing joint strength, and their overall effects on the 95%ile strength value were quantified. Plywood thickness at the 95%ile strength level did not greatly influence joint strength, and has not been included as a variable here. In order to account for these variables the overstrength factor $\phi_o$ may be expressed as

$$\phi_o = K_d K_w K_t (1.461 + 0.054 l_n - 0.724 d_n) \quad (3.30)$$

where the loading rate factor:

$$K_d = \begin{cases} 
1.0 & \text{for quasi-static loading} \\
1.2 & \text{for dynamic loading rate associated with earthquakes}
\end{cases}$$

and the moisture content factor:

$$K_w = \begin{cases} 
1.0 & \text{for green/dry timber at 18%} \\
1.2 & \text{for green/dry timber at 10%} \\
1.4 & \text{for dry/dry timber at 10%}
\end{cases}$$
and the nail type factor:

\[ K_t = \begin{cases} 
1.0 & \text{for plain bright steel or zinc coated nails} \\
1.4 & \text{for galvanised nails.} 
\end{cases} \]

3.8. **CONCLUSIONS**

1. The experimental results demonstrate that more stable ductile joint behaviour occurs when the nail point withdraws from the framing, as opposed to the nail head pulling through the sheathing. This is because there is a sudden loss of load carrying capacity when nail head pull-through occurs. Further experimental studies are required to define the limits on joint geometry necessary to enforce nail point withdrawal. However, it is unlikely that jolt head nails are capable of enforcing nail point withdrawal when incorporated in sheathed framing joints.

2. The distribution of joint ultimate strength is best modelled by the three parameter Weibull distribution. However, the 95\%ile characteristic strength value can be estimated within 2\% of its nonparametric value by assuming a normal distribution.

3. The most significant variables influencing the bearing stress-embedment characteristics of Pinus Radiata were in descending order of importance: timber density, moisture content, and nail diameter. For the 1.8 to 3.3 mm diameter nails considered, nail diameter is not a significant variable. A small reduction in bearing stress with increasing nail diameter was observed for loading parallel to the grain.

4. An ultimate strength model for nailed joints has been developed. Using the Monte Carlo simulation technique, in conjunction with the known distributions of the material properties, the model can be used to predict the 5\%ile,
50\%ile and 95\%ile characteristic joint strengths. The 95\%ile prediction is shown to be highly dependent on the withdrawal resistance of the nail shank.

5. The ultimate strength of a nailed joint is shown to be dependent on the bearing strength of the timber, plastic moment of the nail, withdrawal resistance of the nail, sheathing thickness, and the coefficient of friction between the joint members. The nail diameter, nail length, and nail coating are identified as being the variables most influential on joint strength.

6. Both the coefficient of friction between timber and nail shank, and the bearing strength of the timber, were dependent on loading rate. A 20\% increase in ultimate strength was predicted when the joint was subjected to dynamic loading.

7. An expression for the overstrength factor $\phi_o$ is proposed to compare the likely overstrength of nailed joints with their respective design earthquake load $P_e$. The value of $\phi_o$ is shown to be dependent on nail diameter, nail length, nail coating, timber moisture content, and loading rate. There is a significant increase in $\phi_o$ with increased nail length, and $\phi_o$ is significantly greater for galvanised as opposed to bright or zinc coated nails.
INTRODUCTION TO EXPERIMENTAL INVESTIGATION OF PLYWOOD SHEATHED SHEAR WALLS

4.1 INTRODUCTION

Although there exists a large body of literature [3] concerning the behaviour of plywood sheathed shear walls, there have been relatively few experimental studies of shear walls subjected to reverse cyclic loading. The findings of several experimental studies considered relevant to this study are summarised in the following section. Where possible experimental behaviour is compared with predictions of the theoretical models presented in Chapter 2.

The experimental study described in this chapter was undertaken with the primary objective of investigating the earthquake performance of plywood sheathed shear walls. Eleven full-scale plywood sheathed shear walls were tested within three loading regimes: four walls were subjected to quasi-static reverse cyclic loading, four walls were subjected to sinusoidal shakeable motion, and three walls were subjected to simulated earthquake shakeable motion. The experimental results from the shakeable tests are used to validate the theoretical time-history shear wall model presented in Chapter 6. The
testing arrangement, design, construction and instrumentation of
the shear walls, and the test procedures are discussed in
Section 4.3.

4.2  PREVIOUS EXPERIMENTAL STUDIES OF SHEAR WALL BEHAVIOUR

In this section, the general experimental behaviour of shear
walls is reviewed briefly, followed by a more detailed summary of
three experimental studies of shear walls considered particularly
relevant to this study. For each of these studies, a description
of the investigation, the results obtained, and comments on some
aspects of the work are made in turn. Concluding remarks
regarding the experimental behaviour of the test walls are made
at the end of this section.

4.2.1  General

Early investigators [12, 13, and 14] carried out experimental
tests on shear walls incorporating standard 1.2 x 2.4 m panels,
and showed that wall strength and stiffness were highly dependent
on the number and strength of the sheathing nailing. Similar
findings for plywood sheathed diaphragms were reported by
Countryman [89].

Currier [91] tested six plywood sheathed shear walls
incorporating strong anchorage connections. He confirmed
previous findings that wall strength was strongly influenced by
the strength of the sheathing nails, and found that wall strength
was approximately proportional to the wall length. Failure of
the test walls occurred either through the nail heads pulling
into the face of the plywood or the nail shank withdrawing from
the framing members. At ultimate load, separation of the framing
members at the framing joints was observed.

Robertson and Griffiths [90] found that the stiffness and
strength of their test walls were not particularly sensitive to
nail spacing and nail strength. The reason that their results
are in conflict with previous findings is due to the type of wall
tested. Their test walls incorporated no specific hold-down connection to transfer vertical uplift forces between the tension framing stud and the test frame. Instead, tension in the edge framing stud was transferred to the bottom framing plate by gusset action of the sheathing across the bottom corner of the panel. The bottom framing plate itself was bolted to the test frame. Consequently, the reported wall stiffness included the effects of rigid body rotations of the test wall, arising from displacements at the lower corners.

4.2.2 Medearis (1966) [92]

4.2.2.1 General Description of Test Programme

One of the first series of tests, investigating the seismic behaviour of shear walls, was carried out by Medearis [92]. Eight full scale tests were reported with test variables being the plywood thickness, nail size, nail spacing, loading history and repair techniques. The primary objective of the study was to establish the energy dissipation and inelastic displacement capacity of the shear walls after they had been subjected to reverse cyclic loading. The test walls were 2.4 m square panels with plywood nailed to one or both sides. Construction details for the test walls are shown in Fig. 4.1. Two cantilever test walls were orientated horizontally, bolted together along their bottom edge, and loaded simultaneously, as illustrated in Fig. 4.1. Strong anchorage connections were provided to resist the anchorage forces between the test walls, as illustrated in Fig. 4.1(b).
Fig. 4.1 Details of Medearis test walls [92].
(a) Testing arrangement.  (b) Anchorage forces.
(c) Construction details.
4.2.2.2 Observed Behaviour

1. All test walls were loaded by reverse cyclic loading with each successive cycle exceeding the load peak of the previous cycle. The recorded load-deflection hysteretic behaviour (i.e., "hysteresis loops") for Test Wall 7 is reproduced in Fig. 4.2. The Fig. 4.2 hysteresis loops represent the average load-deflection behaviour for the two walls tested in the arrangement shown in Fig. 4.1. The cyclic behaviour is characterised by progressive pinching of the hysteresis loops during successive cycles. The tangent stiffness at small displacements degrades with each ensuing cycle, but increases as the displacements approach the previous cyclic peak. The shape of the hysteresis loops seen in Fig. 4.2 is typical of all the tested walls. Maximum total wall displacements $\Delta_T$ of between 40 and 50 mm were obtained prior to failure of the test walls.

![Graph showing load-deflection relationship for Test Wall 7](image)

**Fig. 4.2** Load-slip hysteresis loops for Test Wall 7 (Medearis [92]).
2. Test Wall 3 was subjected to twenty load cycles at design load prior to the testing procedure described above. Comparison of the load-deflection behaviour of Test Wall 2 and 3, which were otherwise identical, suggests that pre-cycling the wall at its design load does not affect ultimate strength.

3. The effect of varying such parameters as the plywood sheathing thickness, nail size, and nail spacing is shown in Fig. 4.3. The parent load-deflection curves indicate an increase in wall strength and stiffness with an increase in plywood thickness, nail size and nailing density.

![Graph showing load-deflection curves for walls tested by Medearis [92].](image-url)
4. Test Wall 5 was repaired after testing by jacking the wall back to its original position and re-nailing (the original nails were not removed); this became Test Wall 6. Test Wall 6 had less stiffness than the original wall during small displacement cycles despite being re-nailed, but had greater stiffness during larger displacement cycles. The reduced stiffness at small displacements was due to the bolt holes in the anchorage connections being enlarged during the previous test. The increased stiffness at large displacements was due to the original nails taking up the slackness within the previously enlarged nail holes and contributing towards the load resistance.

5. Failure of the test walls generally occurred when the sheathing nails pulled out from the framing or pulled through the sheathing. In Test Wall 4 a brittle shear failure of the plywood sheathing occurred, initiating at the bolts of the hold-down connection (see Fig. 4.1).

4.2.3 **Thurston and Flack (1980) [33]**

4.2.3.1 General Description of Test Programme

Thurston and Flack [33] tested five full scale particle board sheathed shear walls. The aim of the study was to investigate their in-plane reverse cyclic response, and determine the effects of fastening the sheathing to the framing with nails and elastomeric adhesive. Construction details for the test walls are shown in Fig. 4.4.

4.2.3.2 Observed Behaviour

1. The overall load-deflection hysteretic behaviour for Test Wall 3 is reproduced in Fig. 4.5. The pinching characteristics evident in the hysteretic loops are similar to those of the plywood sheathed test walls reported by Medearis [92] (Fig. 4.2).
Fig. 4.4 Details of Thurston and Flack [33] test walls.
(a) Testing arrangement. (b) Construction details.
Fig. 4.5  Load-slip hysteresis loops for Test Wall 3 (Thurston and Flack [33]).

2. Horizontal deflections $\Delta_a$ arising from anchorage deformations (Fig. 2.3) contributed up to 20% of the total horizontal deflection $\Delta_T$. Anchorage connection deformations were 3 to 5 mm at the base horizontal shear connection; 1 to 3 mm at the tension hold-down connection; and 5 to 7 mm bearing deformation under the compression framing stud.

3. The ultimate strength of Test Wall 3 was 2.2 times higher than the permissible NZS 3603:1981 wind/seismic load $V_{WS}$ of 31.2 kN. The NZS 3603:Table 29 expression for shear wall deflection provided a good estimate of the measured lozenzing stiffness of Test Wall 3.

4. Test walls fabricated with elastomeric adhesive exhibited similar initial stiffness to, but higher ultimate strengths than the other test walls. The
nailed test walls without elastomeric adhesive (Test Walls A, 2 and 4) exhibited improved ductile behaviour from those walls (Test Walls 1 and 3) incorporating elastomeric adhesive.

5. Three failure modes were identified:

(a) Brittle failure of the edge framing stud member occurred in Test Wall A. A hold-down connection was positioned on the outside and inside faces of the edge framing stud (see Fig. 4.4) for all the test walls except Test Wall A, in which one connection was positioned on the outside face of the edge framing stud. The eccentricity of the applied hold-down tension force acting on the framing stud caused bending moment to be superimposed on the axial tension force; leading to the brittle failure.

(b) Brittle shear failure of the particle board sheathing occurred in Test Walls 3 and 4.

(c) Ductile failure of the nail fasteners occurred in Test Walls 1 and 2.

4.2.4 Thurston (1984) [34]

4.2.4.1 General Description of Test Programme

Thurston subjected 10 full scale plywood sheathed shear walls to reverse cyclic loading. Test variables included wall geometry, nail size, framing joint connection detail, and loading rate. Two 2.4 m high by 3.6 m long walls and eight 2.4 m high by 1.2 m long walls were tested. Details of the shear walls and the strong anchorage connections used are shown in Fig. 4.6.
(d)

Fig. 4.6 Details of Thurston [34] test walls. (a) General arrangement of 3.6 m long walls. (b) General

<table>
<thead>
<tr>
<th>TEST WALL</th>
<th>SIZE</th>
<th>SHEATHING WAILING TYPE</th>
<th>SPACING (mm)</th>
<th>PLYWOOD THICKNESS (mm)</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.4 x 1.2</td>
<td>50 x 2.5</td>
<td>50</td>
<td>9 One side</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>2.4 x 1.2</td>
<td>50 x 2.5</td>
<td>50</td>
<td>9 One side</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2.4 x 1.2</td>
<td>30 x 3.15</td>
<td>50</td>
<td>9 One side</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>2.4 x 1.2</td>
<td>30 x 3.15</td>
<td>50</td>
<td>9 One side</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2.4 x 1.2</td>
<td>30 x 3.15</td>
<td>50</td>
<td>9 Both sides</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>2.4 x 1.2</td>
<td>30 x 3.15</td>
<td>50</td>
<td>9 One side</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>2.4 x 1.2</td>
<td>30 x 3.15</td>
<td>50</td>
<td>9 One side</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>2.4 x 1.2</td>
<td>30 x 3.15</td>
<td>50</td>
<td>9 One side</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>2.4 x 1.2</td>
<td>30 x 3.15</td>
<td>50</td>
<td>9 Both sides</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>2.4 x 1.2</td>
<td>30 x 3.15</td>
<td>50</td>
<td>9 One side</td>
<td></td>
</tr>
</tbody>
</table>

Wall 1 retested with stiffer S/D connections
Dynamic Loading
Identical to Wall 4
Edge stud extended full height
Edge stud extended full height

(c) Detailed view of...

(b) General arrangement of 3.6 m long walls.

(a) General arrangement of 3.6 m long walls.
4.2.4.2 Observed Behaviour

1. The observed pinching hysteretic response was similar to that obtained in the test walls reported by Thurston and Flack [33] and Medearis [92].

2. Horizontal deflection $\Delta_A$ arising from measured anchorage displacements accounted for more than 40% of the total horizontal deflection $\Delta_T$ of the 1.2 m long walls, and only 10% of the total horizontal deflection $\Delta_T$ of the 3.6 m long walls. The significant difference in $\Delta_A$ for the two wall types is due to the effect of wall geometry on rigid body rotations which arise from vertical anchorage displacements (see Fig. 2.3). Because of the position of the deflection gauge (see Fig. 4.6), the measured vertical anchorage displacements may have been less than the actual displacements.

3. Separation of the top framing plate and framing stud connections occurred as the ultimate strength $V'$ was approached. From Section 2.4, the predicted framing joint forces are $X' = 0.13 V'$ and $Y' = 0.08V'$, and these exceeded the capacity of the two 150 x 6.0 mm nails provided as the connection between the framing members. Nail plate reinforcement in Test Wall XS was found to be the only effective means of transferring the joint forces and controlling framing member separation.

4. The strength of Test Walls 5 and B (which had sheathing fastened to both sides of the framing members) was limited to the bearing strength of the bottom framing plate under the compression framing stud. Consequently, Test Wall 5 was only 25% stronger than Test Wall 3, despite being sheathed on both sides and therefore having twice the number of sheathing nails.

5. The ultimate strength of the test walls, excluding those whose strength was limited by the bearing strength of
the bottom framing plate, was at least 2 times greater than their respective NZS 3603:1981 permissible wind/seismic load $V_{ws}$.

6. Comparison of the lozenging load-deflection response shows that the 3.6 m long walls had approximately three times the stiffness and strength of the otherwise identical 1.2 m long walls.

7. Three failure modes were identified:

(a) The sheathing nails withdrew from the framing in seven of the ten test walls.

(b) Nail force components acting in the direction perpendicular to the framing caused local perpendicular-to-grain tension failure of the bottom framing plate. This occurred in the 3.6 m long Test Walls 6 and 7 during the 60 mm displacement cycle.

(c) Brittle shear failure of the plywood occurred in Test Wall XS.

4.2.5 Conclusions from Previous Experimental Studies

1. Plywood and particle board sheathed shear walls are capable of sustaining large horizontal displacements. This is due to the large slip capacity of the sheathing nails. This preferred response, however, may be inhibited through failure of the sheathing, framing members, or anchorage connections. During repeated reverse cyclic loading the load-deflection behaviour of the wall is characterised by pinched hysteresis loops.

2. Anchorage connections can control stiffness and strength of shear walls. Horizontal wall deflection arising from bearing deformations in the bottom framing plate under the compression framing stud may be significant.
3. A steel angle bracket or nail plate connection recessed into the framing under the sheathing may be required to resist the framing joint forces.


**Table 4.1 Description of Shear Walls.**

<table>
<thead>
<tr>
<th>Material</th>
<th>WALL (1)</th>
<th>WALL (3)</th>
<th>WALL (6) &amp; (7)</th>
<th>WALL (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheathing</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$H$ (mm)</td>
<td>2400</td>
<td>2400</td>
<td>2400</td>
<td>2400</td>
</tr>
<tr>
<td>$B$ (mm)</td>
<td>1200</td>
<td>1200</td>
<td>1200</td>
<td>1200</td>
</tr>
<tr>
<td>$n$</td>
<td>1</td>
<td>1</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>$E_x$ (MPa)</td>
<td>10,800</td>
<td>10,800</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_y$ (MPa)</td>
<td>10,800</td>
<td>10,800</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$G$ (MPa)</td>
<td>950</td>
<td>950</td>
<td>950</td>
<td>1400</td>
</tr>
<tr>
<td>$v$</td>
<td>0.3*</td>
<td>0.3*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$t_s$ (mm)</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>Framing</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E$ (MPa)</td>
<td>8000</td>
<td>8000</td>
<td>8000</td>
<td>8000</td>
</tr>
<tr>
<td>$b$ (mm)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>$a$ (mm)</td>
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<td>100</td>
<td>100</td>
<td>150</td>
</tr>
<tr>
<td>Sheathing nails</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P_0$ (N)</td>
<td>625</td>
<td>940</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P_1$ (N/mm)</td>
<td>104</td>
<td>2000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$k$ (N/mm)</td>
<td>1136</td>
<td>62.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c$ (mm)</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Framing Joins</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P_{0,x}$ (N)</td>
<td>8800*</td>
<td>8800*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P_{1,x}$ (N/mm)</td>
<td>0*</td>
<td>0*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$k_x$ (N/mm)</td>
<td>8800*</td>
<td>8800*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P_{0,y}$ (N)</td>
<td>3000*</td>
<td>3000*</td>
<td></td>
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</tr>
<tr>
<td>$P_{1,y}$ (N/mm)</td>
<td>0*</td>
<td>0*</td>
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<td></td>
</tr>
<tr>
<td>$k_y$ (N/mm)</td>
<td>3000*</td>
<td>3000*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P_{1,\theta}$ (N)</td>
<td>0*</td>
<td>0*</td>
<td></td>
<td></td>
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<td>$P_{1,\theta}$ (N/mm.rad)</td>
<td>0*</td>
<td>0*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$k_{\theta}$ (N/mm.rad)</td>
<td>0*</td>
<td>0*</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*estimated values only
4.2.6 **Comparison of Results from the Experimental Tests [33,34]**
and Predictions of the Proposed Shear Wall Model
Described in Chapter 2

In this section, the results from two of the experimental studies [33,34] just reviewed are compared with predictions made by finite element analysis [24], and with predictions made by the shear wall theories advanced in Chapter 2. The two experimental studies were chosen for the comparison because companion sheathing joint tests (similar to that shown in Fig. 2.11) were included.

4.2.6.1 **Predicted Elastic Behaviour Based on the Load-Slip Behaviour of Separate Sheathing Nailed Joint Tests**

In this section, the load-deflection response of the test wall is predicted, using the load-slip characteristics of the matching nailed sheathing joint tests.

Details of the four test walls used for the comparison, and the material properties required as input data for the finite element analysis and the proposed elastic shear wall model, are tabulated in Table 4.1. Parameters \( P_0 \), \( P_1 \), \( k \) for the nailed framing joints, and Poisson's ratio \( \nu \) for the sheathing were estimated. All other material properties listed in Table 4.1 were obtained from tests on the actual materials used in the Test Walls, as reported in the literature [33, 34]. The experimental load-slip characteristics of the nailed joint tests are listed in Table 4.2. The Table 4.2 load-slip results enable the average nail slip \( \Delta_{av} \) to be determined for calculated average nail force \( P_{av} \) values (Eq. 2.44), as described in Section 2.3.7.
Table 4.2  Load-slip Characteristics of Nailed Sheathing Joints.

<table>
<thead>
<tr>
<th>Nail slip (mm)</th>
<th>60 x 3.15</th>
<th>30 x 3.15</th>
<th>50 x 2.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>490</td>
<td>360</td>
<td>360</td>
</tr>
<tr>
<td>0.4</td>
<td>630</td>
<td>530</td>
<td>340</td>
</tr>
<tr>
<td>0.8</td>
<td>810</td>
<td>720</td>
<td>480</td>
</tr>
<tr>
<td>1.6</td>
<td>1000</td>
<td>930</td>
<td>670</td>
</tr>
<tr>
<td>3.2</td>
<td>1190</td>
<td>1130</td>
<td>930</td>
</tr>
<tr>
<td>4.8</td>
<td>1330</td>
<td>1240</td>
<td>1120</td>
</tr>
<tr>
<td>6.4</td>
<td>1380</td>
<td>1310</td>
<td>1220</td>
</tr>
</tbody>
</table>

Parent load-slip curves were constructed from the envelope of the positive experimental hysteresis loops. Figure 4.7 shows the experimental parent curve, the theoretical predictions given by the finite element analysis, and the proposed theory. There is good agreement between the test results and the theory up to horizontal lozenging deflections of 15 mm.

4.2.6.2 Predicted Load-Deflection Behaviour Based on Average Nail Slip Measured During Shear Wall Tests

In this section, the average sheathing nail slip $\Delta_{av}$ required for the Table 2.1 shear wall deflection expression was obtained from the actual nail slip values measured during the shear wall test corresponding to each applied load level. The sheathing nail slip was measured parallel to the sheathing edge at four locations around each panel (see Fig. 4.8), and averaged to give the parent wall load versus nail slip curve, shown in Fig. 4.8. In this way, the proposed theory is compared directly with the experimental results.
Fig. 4.7  Comparison of results from the proposed Elastic shear wall model with those from tests and finite element analysis.
Fig. 4.7  Continued.
Fig. 4.8 Typical wall load versus average nail slip hysteresis loops.

Figure 4.9 compares the experimental (positive and negative) parent load vs lozenging deflection curves with the proposed model predictions. Very good agreement exists between the theory and the experimental load-declection curves.

4.2.6.3 Predicted Shear Wall Strength

Table 4.3 tabulates the theoretical wall ultimate strength (see Table 2.10) and the experimental wall strengths. Generally good agreement exists between experiment and theory. The discrepancy between the theoretical and experimental ultimate strength of Test Wall 5 (Thurston 1984 test series) is due to the test wall ultimate strength being controlled by yielding anchorage connections, rather than the load-slip characteristics of the sheathing nails, as discussed in Section 4.2.4.2.
Fig. 4.9 Comparison of results from proposed shear wall model with those from tests.
### Table 4.3 Comparison of Predicted and Experimental Wall Strengths.

<table>
<thead>
<tr>
<th>Test Wall</th>
<th>Paired joint tests</th>
<th>$V_o = \frac{1.15 P_o n B}{c}$</th>
<th>Experimental</th>
<th>Theory Experiment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall (1) 1980 [33]</td>
<td>1.650</td>
<td>90</td>
<td>76</td>
<td>1.18</td>
</tr>
<tr>
<td>Wall (3) 1980 [33]</td>
<td>1.650</td>
<td>90</td>
<td>76</td>
<td>1.18</td>
</tr>
<tr>
<td>Wall (2) 1984 [34]</td>
<td>1.220</td>
<td>33</td>
<td>30</td>
<td>1.10</td>
</tr>
<tr>
<td>Wall (3) 1984 [34]</td>
<td>1.310</td>
<td>36</td>
<td>35</td>
<td>1.03</td>
</tr>
<tr>
<td>Wall (5) 1984 [34]</td>
<td>1.310</td>
<td>72</td>
<td>42</td>
<td>1.71</td>
</tr>
<tr>
<td>Wall (6) 1984 [34]</td>
<td>1.310</td>
<td>107</td>
<td>88</td>
<td>1.22</td>
</tr>
<tr>
<td>Wall (7) 1984 [34]</td>
<td>1.310</td>
<td>107</td>
<td>100</td>
<td>1.07</td>
</tr>
</tbody>
</table>

#### 4.3 Design and Construction of Test Walls

**4.3.1 General**

The test walls were designed to be representative of strong shear walls typical of those used in commercial construction. Consequently, the test walls incorporate smaller nail spacing, larger framing members, and stronger anchorage connections than shear walls commonly used in conventional light frame timber construction.

The test walls were grouped into three test series, as summarised in Table 4.4. Test Walls S1 to S4 were subjected to quasi-static reverse cyclic loading. The purpose of this first series was to determine load-deflection behaviour and failure modes, and to closely monitor the behaviour of the framing members, sheathing nails, and anchorage connections. Prior to quasi-static loading, Test Walls S1 and S4 were subjected to resonant testing in order to establish their elastic damping properties. This involved repeated small amplitude sinusoidal shakeable tests over a range of shakeable frequencies either side of the natural frequency of the test wall. The resulting wall displacements were small, and the wall response was essentially elastic.
Sinusoidal shaketable tests of Test Walls S5 to S8 formed the second test series. The primary aim of this test series was to validate the analytical dynamic shear wall model. Results also enabled the magnitude of damping within the inelastic range of the wall response, and the effect of loading rate on wall stiffness and strength, to be determined. Sinusoidal shaketable motion was adopted, as it permitted more accurate analytical modelling of the experimental results than what an actual earthquake ground motion record would allow.

The objective of Test Walls S9 to S11 (Table 4.4) was to investigate shear wall performance during an actual earthquake ground motion record. The effects of increasing slackness in the load-deflection hysteretic behaviour of the shear walls during earthquake attack was of particular interest.

Table 4.4 Test Walls.

<table>
<thead>
<tr>
<th>TEST SERIES</th>
<th>TEST WALL</th>
<th>LOADING</th>
<th>Sheathing thickness (mm)</th>
<th>Sheathing nail spacing (mm)</th>
<th>Permissible Wind/Seismic NZS3603:1981 Design load $V_{w, s}$ (kN)</th>
<th>NZS4203:1961 Design Earthquake load $V_E$ (kN)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Quasi Static</td>
<td>7.5</td>
<td>75</td>
<td></td>
<td>18.7</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>1</td>
<td>S2</td>
<td>&quot;</td>
<td>7.5</td>
<td>150</td>
<td>9.4</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>&quot;</td>
<td>&quot;</td>
<td>7.5</td>
<td>75</td>
<td>18.7</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>S4</td>
<td>&quot;</td>
<td>&quot;</td>
<td>12</td>
<td>75</td>
<td>18.7</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>S5</td>
<td>Shaketable Sinusoidal</td>
<td>7.5</td>
<td>75</td>
<td></td>
<td>18.7</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>2</td>
<td>S6</td>
<td>&quot;</td>
<td>7.5</td>
<td>75</td>
<td>18.7</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>S7</td>
<td>&quot;</td>
<td>&quot;</td>
<td>7.5</td>
<td>75</td>
<td>18.7</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>S8</td>
<td></td>
<td>&quot;</td>
<td>12</td>
<td>75</td>
<td>18.7</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>S9</td>
<td>Shaketable El Centro 1940</td>
<td>7.5</td>
<td>75</td>
<td></td>
<td>18.7</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>3</td>
<td>S10</td>
<td>&quot;</td>
<td>7.5</td>
<td>75</td>
<td>18.7</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>S11</td>
<td></td>
<td>&quot;</td>
<td>7.5</td>
<td>150</td>
<td>9.4</td>
<td>12</td>
<td></td>
</tr>
</tbody>
</table>
Fig. 4.10  Construction details of test walls.
The test walls were designed in accordance with the NZS3603:1981 procedures as described in Chapter 2 (Table 2.1). Figure 4.10 shows the construction details of the test walls. All walls were 2.4 m square, framed with 150 x 50 mm dry green gauged Pinus Radiata, and sheathed on one side with plywood. The plywood was fixed to the framing around its perimeter with 50 x 2.89 mm diameter zinc coated gun 'T' nails at 75 and 150 mm spacing. The plywood was fixed to the internal framing members at 150 mm spacing. Details of the plywood, framing and nails are presented in Table 4.5.

Within each test series the sheathing nail spacing, framing connection detail, sheathing thickness, and hold-down connections were varied, as detailed in Table 4.4. The 7.5 mm thick plywood sheathing represents the minimum plywood thickness permitted by NZS 3615:Table 17 [73] for 2.8 mm diameter nails. Test Walls S4 and S8 were fabricated with 12 mm thick plywood in order to confirm the results from the nailed joint tests reported in the previous chapter, which showed that nails driven through the thicker plywood had improved ductile characteristics.

The simple diaphragm theory, on which the NZS3603:1981 design procedure is based, ignores the nail force component acting in the direction perpendicular to the framing members, and therefore, neglects the framing joint forces X and Y. The two 100 x 4.0 mm diameter nails connecting the framing members

<table>
<thead>
<tr>
<th>Supply</th>
<th>Grade</th>
<th>Size</th>
<th>Nominal Density Mean (kg/m²)</th>
<th>Moisture Content Mean (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.5 mm Plywood</td>
<td>NZ Forest Products</td>
<td>D-D</td>
<td>1.2 x 2.4 x 7.0</td>
<td>480</td>
</tr>
<tr>
<td>12 mm Plywood</td>
<td>NZ Forest Products</td>
<td>D-Cp Sanded one side</td>
<td>1.2 x 2.4 x 12.2</td>
<td>488</td>
</tr>
<tr>
<td>150 x 150 framing</td>
<td>Nelson grown</td>
<td>No. 1 framing</td>
<td>145 x 45</td>
<td>442</td>
</tr>
<tr>
<td>2.89 mm nails</td>
<td>Able staples</td>
<td>-</td>
<td>50 x 2.89 Zinc Coated 'T' nails</td>
<td>-</td>
</tr>
</tbody>
</table>
(Fig. 4.10) were provided in these test walls because they represent the connection detail commonly used in practice, as detailed in NZS 3604:1981, "Code of Practice for Light Timber Framed Buildings not Requiring Specific Design" [93]. The two nails were not specifically designed to resist the joint forces X and Y predicted by the theory presented in Chapter 2. The joint forces X and Y exceeded the strength of the nail pair during testing of Walls S1 and S2, and therefore the framing joints of Test Walls S3 and S4 were reinforced with nailed steel brackets prior to testing.

The majority of the test walls incorporated strong anchorage connections to resist the uplift forces at the end of the edge framing stud, and the shear force along the framing plate, (see Fig. 2.1). The connections were designed to be capable of developing the strength of the sheathing nails. This ensured that the majority of the total wall deflection $\Delta_T$ was a result of wall lozenging rather than rigid body rotation arising from anchorage displacements. Test Wall S7 was specifically designed with weaker anchorage connections so that the influence of anchorage flexibility on the wall dynamic response could be investigated. For a similar reason, initial slackness was provided in the vertical anchorage connections of Test Walls S6 and S9 to represent the actual gaps between the framing and support beams that would arise after shrinkage of the timber after fabrication.

To permit unrestrained slip over the framing, a 2 mm gap was provided between adjoining plywood sheathing panels, and the framing joists were recessed 2 mm inside the inner face of the plywood, as illustrated in Fig. 4.10. The permissible wind/seismic lateral load $V_{WS}$ in Table 4.4 was calculated in accordance with NZS3603:1981 procedures, where the permissible wind/seismic nail load $P_{WS}$ for the 50 x 2.89 mm diameter nails is given in NZS 3603:Table 28 as being independent of plywood thickness.

The test walls were subjected to inplane reverse cyclic loads. No vertical load was applied to the test wall, and this was
intended to reflect the relatively low vertical loading supported by many actual shear walls. Often the gravity loading is shared among many walls or carried independently by gravity resisting frames. Furthermore, the no-load case was considered the most onerous loading condition for the hold-down connections, which resist uplift forces when the wall is subjected to lateral load, (Fig. 2.1).

![Graph showing shear stress-strain relationship](image)

**Fig. 4.11** Shear stress-strain regression curve for 7.5 mm thick plywood.

**Table 4.6** Test Results for Modulus of Rigidity for 7.5 mm thick Plywood.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Nominal Density</th>
<th>Moisture Content</th>
<th>Thickness</th>
<th>Modulus of Rigidity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kg/m³</td>
<td>%</td>
<td>mm</td>
<td>MPa</td>
</tr>
<tr>
<td>A</td>
<td>478</td>
<td>9.4</td>
<td>7.8</td>
<td>853</td>
</tr>
<tr>
<td>B</td>
<td>499</td>
<td>8.9</td>
<td>7.8</td>
<td>883</td>
</tr>
<tr>
<td>C</td>
<td>486</td>
<td>10.0</td>
<td>7.6</td>
<td>803</td>
</tr>
</tbody>
</table>

7.7 ave 846 ave
4.3.2 Modulus of Rigidity of Plywood Sheathing

The modulus of rigidity of the 7.5 mm thick plywood was determined using an adaptation of the ASTM D2719:Method B [99] test rig, as shown in Plate 4.1. Three test specimens were fabricated 450 mm square, with 100 x 30 mm rails glued around the perimeter of each plywood face (see Plate 4.1). The linear strain across a diagonal was measured with a linear potentiometer (350 mm gauge length) mounted on each side of the specimen. The shear stress-strain curve for test specimen A is shown in Fig. 4.11. Table 4.6 tabulates the modulus of rigidity, moisture content, and Nominal Density of the three specimens tested.

![Graph showing load-slip curve for nailed plywood joint.](image)

Fig. 4.12 Load-slip curve for nailed plywood joint.

4.3.3 Sheathing Nail Load-Slip Characteristics

Ten nailed sheathing joint specimens were fabricated with the test walls using the same framing, sheathing, and pneumatically driven gun nails. Details of the joint design are shown in Fig. 3.15. The joints were stored with the test walls until testing. Monotonic load was applied to the test joints using the loading apparatus shown in Plate 3.1. Relative slip between the
sheathing on one face of the test joint and one framing rail was measured by a linear potentiometer mounted at the end of the nail group. Figure 4.12 shows the mean load-slip curve and the coefficient of variation for the test joints. The initial portion of the load-slip curve, however, may not truly represent the average load-slip behaviour of all the nails in the joint. An investigation of joint behaviour at the end of testing showed that the slip of the plywood on the opposite face of the test joint differed by as much as 0.5 mm from that measured. The errors would have been most significant at small joint slips.

4.3.4 Anchorage Connection Behaviour

The hold-down connection detail (see Fig. 4.10) was selected after preliminary testing of hold-down connections commonly used in practice. The testing was undertaken to ensure that the selected detail would have sufficient strength to develop the strength of the sheathing nails.

![Diagram](image)

**Fig. 4.13** Testing arrangement for hold-down connection test units.
(a) UNIT H3: Perpendicular to the grain tension failure of bottom framing plate.

(b) UNIT H5: timber within the perimeter of the nail plate pulled away from remainder of the framing stud.

Plate 4.1 Plywood Modulus of Rigidity Test.

Plate 4.2 Failure Modes of Hold-down Connection.
4.3.4.1 Results from the Preliminary Experimental Study

Eight hold-down connection test units were fabricated and subjected to reverse cyclic loading. Each of the eight units consisted of a pair of similar hold-down connections mounted in the testing apparatus shown in Fig. 4.13. The framing was 150 x 50 mm dry green gauged Pinus Radiata, having average Nominal Density of 405 kg/m$^3$ at 12.3% moisture content. Details of the test units together with design loads are summarised in Table 4.7. The tabulated test unit loads in Table 4.7 should be halved to obtain the corresponding loads for each hold-down connection.

The permissible wind/seismic load for each component within the test unit was determined using NZS 3603:1981 [11], AS 1250:1975 [96], and nail plate manufacturer's [97] criteria. The size of the bearing washers in Test Units H1 and H2, and the channel bracket in Test Units H4 and H7 were proportioned to ensure that the NZS:3603:1981 [11] permissible perpendicular-to-grain bearing stresses were not exceeded in the framing. The permissible wind/seismic load $P_{H,ws}$ for each test unit was taken as the least of the tabulated nail, nail-plate, bolt or tie, permissible wind/seismic load. In some test units (see Table 4.7), the eccentricity of the applied force acting on the framing stud caused bending moment to be superimposed on the axial tension force. Permissible loads for combined bending and tension of the framing studs are also tabulated in Table 4.7. However, this limitation was disregarded in determining the permissible seismic load $P_{H,ws}$ at the time of testing, as framing flexural stresses are usually neglected in shear wall design as being insignificant [11,102].

Four reverse cycles of load were imposed on the test unit at increasing displacement amplitudes, as illustrated by the load-deflection hysteresis loops of Test Unit H1 shown in Fig. 4.14.
### Table 4.7: Design Details and Failure Loads of Hold-Down Connection Test Units

<table>
<thead>
<tr>
<th>Component</th>
<th>Stud*</th>
<th>Nails</th>
<th>Nail Plate</th>
<th>M16 bolt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible Seismic Load (kN)</td>
<td>27</td>
<td>40</td>
<td>45</td>
<td>66</td>
</tr>
<tr>
<td>Permissible Seismic Load for unit $P_{H,ws}$</td>
<td>40 (kN)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Load (kN)</td>
<td>120; 3.0 $P_{H,ws}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Brittle tearing failure along the bend of the nail plate.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>Stud*</th>
<th>Nails</th>
<th>Nail Plate</th>
<th>M12 bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible Seismic Load (kN)</td>
<td>27</td>
<td>40</td>
<td>45</td>
<td>74</td>
</tr>
<tr>
<td>Permissible seismic load for unit $P_{H,ws}$</td>
<td>40 (kN)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Load (kN)</td>
<td>115; 2.9 $P_{H,ws}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Brittle tearing failure along the bend of the nail plate.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>Stud</th>
<th>Nails</th>
<th>Nail Plate</th>
<th>M12 bolt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible Seismic Load (kN)</td>
<td>70</td>
<td>22</td>
<td>90</td>
<td>24</td>
</tr>
<tr>
<td>Permissible seismic load for unit $P_{H,ws}$</td>
<td>22 (kN)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Load (kN)</td>
<td>23; 1.0 $P_{H,ws}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Splitting of framing plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(see Plate 4.2(a))</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>Stud*</th>
<th>M16 bolt</th>
<th>12 mm Ø tie</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible Seismic Load (kN)</td>
<td>13</td>
<td>21</td>
<td>17</td>
</tr>
<tr>
<td>Permissible seismic load for unit $P_{H,ws}$</td>
<td>21 (kN)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Load (kN)</td>
<td>63; 3.0 $P_{H,ws}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Brittle failure of framing stud (corresponded to 30 MPa extreme tension fibre stress)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Combined bending and tension
** Least of tabulated nail, nail-plate, bolt or $\frac{1}{2}w$ code
Table 4.7  Continued.

<table>
<thead>
<tr>
<th>Component</th>
<th>Stud$^*$</th>
<th>Nails</th>
<th>Nail Plate</th>
<th>16 mm Ø tie</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible Seismic load (kN)</td>
<td>27</td>
<td>40</td>
<td>45</td>
<td>66</td>
</tr>
<tr>
<td>Permissible Seismic load for unit $P_{H,ws}$**</td>
<td>40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure load (kN)</td>
<td>125; 3.1 $P_{H,ws}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Timber within the perimeter of the nail plate pulled away from stud (see Plate 4.2(b))</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>Stud$^*$</th>
<th>Nails</th>
<th>Nail Plate</th>
<th>16 mm Ø tie</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible Seismic load (kN)</td>
<td>27</td>
<td>22</td>
<td>23</td>
<td>66</td>
</tr>
<tr>
<td>Permissible Seismic load for unit $P_{H,ws}$**</td>
<td>22</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure load (kN)</td>
<td>95; 4.3 $P_{H,ws}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Timber within the perimeter of the nail plate pulled away from the stud (see Plate 4.2(b))</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>Stud$^*$</th>
<th>M20 bolts</th>
<th>12 mm Ø tie</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible Seismic load (kN)</td>
<td>13</td>
<td>65</td>
<td>37</td>
</tr>
<tr>
<td>Permissible Seismic load for unit $P_{H,ws}$**</td>
<td>37</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure load (kN)</td>
<td>60; 1.6 $P_{H,ws}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Brittle failure of framing stud. (corresponded to 30 MPa extreme tension fibre stress)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>Stud</th>
<th>Nails</th>
<th>Nail Strap</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible Seismic load (kN)</td>
<td>70</td>
<td>29</td>
<td>16</td>
</tr>
<tr>
<td>Permissible Seismic load for unit $P_{H,ws}$**</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure load (kN)</td>
<td>32; 2.0 $P_{H,ws}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Brittle tensile failure of nail strap.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Combined bending and tension
** Least of tabulated nail, nail-plate, bolt or tie loads.
Detailed results of the load-deflection behaviour of each unit are fully described elsewhere [98]. For brevity, only a summary of the major findings is given below:

1. Table 4.7 summarises the failure loads and failure modes of the eight test units. Most test units attained an ultimate strength of at least $2\frac{1}{2}$ times the permissible wind/seismic load $P_{H,ws}$. All test units exhibited a brittle failure mode either through failure of the timber, or failure of the steel nail plates or straps. Brittle failure at the bend of the 2 mm thick nail plate led to the failure of Test Units H1 and H2. Tensile tests of the 2 mm thick steel nail plate and the 1 mm thick nail strap (Test Unit H8), showed that the steel remained essentially elastic up until failure of 370 MPa and 700 MPa respectively. The rigid steel brackets incorporated in Test Units H4 and H7 induced large eccentricities between the tie force and the framing stud. The combined bending and tension forces resulted in a brittle failure of the framing stud. The low eccentricity of the hold-down force with the framing stud is a feature of the preferred detail of Test Units H5 and H6. Failure of these test units occurred when all the timber within the perimeter of the nail-plate pulled away from the remaining framing stud, as shown in plate 4.2 (b). Test Unit H3 performed particularly poorly with the lower framing plate splitting when the joint load reached $1.0 P_{H,ws}$, as shown in Plate 4.2(a).

The performance of this detail would be improved if the nail plate was wrapped under the framing plate and nailed to both sides of the framing stud.
Fig. 4.14 Load-deflection hysteresis loops for Test Unit H1.

2. The load-deflection hysteresis loops for unit H1 are reproduced in Fig. 4.14. The hysteresis loops seen in Fig. 4.14 are typical of the behaviour observed in all eight test units. In tension, the hysteresis loops exhibit pinching characteristics similar to that observed in the nailed joints reported in Section 3.3. In compression, the hysteresis loops remain essentially elastic up to loads between 80 and 100 kN. The compressive strength of the test unit was developed through perpendicular-to-the-grain bearing, aided by the strength of the nail plates in Test Units H1 to H3 and H9. Test Units H4 and H7 incorporated bolts and their hysteresis loops exhibited initial slackness due to slightly oversized bolt holes. This initial slackness may lead to unacceptable free movement of the structure under serviceability conditions.
3. When the test units were subjected to tension loading, separation between the framing stud and the framing plate exceeded 10 mm in some cases. Separation of this magnitude is likely to lead to local damage of the sheathing or the sheathing nails at the bottom corner of the shear wall. This could be inhibited in details such as Units H1, H2 and H4 to H7 (see Table 4.7) if the lower framing plate is permitted to rise with the framing stud and sheathing.

4.3.4.2 Anchorage Connections for Test Walls

The anchorage connections of all test walls except Test Wall S7 were designed to be capable of developing the strength of the sheathing nails. The maximum likely ultimate strength of the sheathing nails was estimated as being 3 times the permissible wind/seismic load $V_{ws}$ of the test wall (see Table 4.4). Both shear and hold-down connections were designed to remain essentially elastic up to a load of $3 V_{ws}$. This ensured that the majority of the total wall deflection $\Delta_T$ would result from lozenging of the wall through nail slip. Figure 4.10 shows the hold-down and shear connection details that were used. The hold-down connection incorporates a 3 mm thick nail-plate welded directly to a 12 mm diameter steel tie. Two connections were mounted symmetrically about each edge framing stud (as shown in Fig. 4.10 (b)) for all test walls except Test Walls S2 and S7. Only one connection, positioned on the outside face of the framing stud, was required to develop the strength of the sheathing nails of Test Wall S2. The hold-down connections of Test Wall S7 were specifically designed to yield before the strength of the sheathing nails had developed, and so only one hold-down connection was used. Table 4.8 tabulates the design loads and estimated strength capacities of the hold-down connections.
Table 4.8 Hold-Down Connection Design Loads and Estimated Strength Capacities.

<table>
<thead>
<tr>
<th>TEST WALLS (kN)</th>
<th>ALLOWABLE SEISMIC LOADS (kN)</th>
<th>EXPECTED CAPACITY (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TENSION</td>
<td>COMPRESSION</td>
</tr>
<tr>
<td></td>
<td>Nails 12 φ Tie Bearing</td>
<td>Nails 12 φ Tie Bearing</td>
</tr>
<tr>
<td>S1, S3 to S6, S8 to S11</td>
<td>30 37 100</td>
<td>88 62 150</td>
</tr>
<tr>
<td>S2 and S7</td>
<td>15 19 100</td>
<td>44 31 150</td>
</tr>
</tbody>
</table>

The shear connection between the framing plate and framing joists consisted of a 1.2 m long nail-plate, as shown in Fig. 4.10. The connection was positioned 600 mm from each end of the wall to permit separation between the framing plate and joists due to uplift at the base of Test Walls S6, S7 and S9 (see Fig. 4.10). This uplift was expected to arise from the slackness within the hold-down connection of Test Walls S6 and S9, and yielding of the hold-down tie of Test Wall S7.

4.3.5 Construction of Test Walls

Eight of the eleven test walls were constructed under normal commercial conditions by Peter Stevens Ltd, Christchurch. The remaining 3 test walls (Test Walls S4, S8 and S11) were fabricated just prior to testing, but using the same framing members as Test Walls S3, S7 and S10 respectively. The sheathing nails were pneumatically driven flush with the plywood surface. A small spacer block ensured that constant nail spacing was maintained. The test walls were stored for about four months prior to testing.
Plate 4.3  Views of Test Assembly.
a) Hold-down Connection

Plate 4.4 Details of Testing Facility.

b) Lateral Support Roller
Emergency stops for inertia mass
Load cell

Hydraulic jack for static tests
Frame
50 x 50 x 5 mm strut to carry axial load
Load cell
Load cell stanchion

Section

Concrete inertia mass
(In 10 t blocks - max 4 t)
Inertia mass support frame

Frame
Channel beam
Base beam
Swing arm
Rollers

Shaketable

Fig. 4.15 General arrangement of testing facility.

Heavy duty castors required to provide sideways restraint
Hydraulic jacks to lift inertia mass between tests
Corbel
Frame
Test specimen
Loading girder
Swing arms
Rollers & packers

End Elevation
4.4 TESTING PROCEDURE

4.4.1 Testing Facility

Quasi-static and dynamic tests were performed using the one testing facility illustrated in Fig. 4.15. Full details of the shaketable and its control system are given elsewhere [99]. Plate 4.3 shows different views of the testing facility. The aerial view (Plate 4.3(a)) shows the test wall, positioned in the bottom base beam and top channel beam, being lifted into the testing facility.

Each test wall was mounted vertically into the testing frame. The bottom joists of the test wall were clamped by bearing plates onto the steel base beam. Overturning forces were transferred from the edge framing studs to the base beam through the nail plate hold-down details, (see Plate 4.4(a)). The base beam was supported on roller assemblies positioned between the beam and the shaketable (Fig. 4.15) to ensure that all of the horizontal thrust applied to the specimen by the shaketable was transferred through the load cell. Vertical thrust on the base beam was transferred to the shaketable through the roller assemblies, and the four swing arm assemblies (Fig. 4.15).

Inplane shear load was transferred to the top of the specimen through the top channel beam, tie rods and nail-on plate assembly, (see Fig. 4.16). This arrangement permitted vertical movement at the top of the wall resulting from displacements of the hold-down connections. A 50 mm gap was provided between the top of the test wall and the channel beam for this. The arrangement also allowed vertical movement, without inducing any significant vertical component of the tie rod force on the wall.

The inertia mass, consisting of four 1000 kg reinforced concrete blocks, was fixed to the top channel beam. High strength steel rods tied the blocks together, and onto a support frame and channel beam. Four pin-ended struts supported the inertia mass
while allowing full inplane movement of the inertia mass and test wall. Thus, no vertical loads were applied to the test wall. In order to prevent collapse of the inertia mass after failure of the test wall, a steel safety frame enclosed the inertia mass. For quasi-static loads, this steel frame acted as a reaction frame for the hydraulic jack positioned between the frame and the top loading channel, as seen in Fig. 4.15. Details of the reaction frame are given in Fig. 4.17.

Lateral support was given to the inertia mass through four heavy duty casters positioned between the mass and the reaction frame, (see Plate 4.4(b)). In this way, only in-plane inertia forces could be applied to the test walls. Lateral support was given to the test wall itself by the channel beam which extended downwards over both sides of the top joists. Frictionless teflon pads were mounted between the joists and the channel beam.

Fig. 4.16 Diagramatic view of loading arrangement.
Fig. 4.17 Details of reaction/safety frame.
4.4.2 Instrumentation

Instrumentation was provided to obtain detailed information on the behaviour of the test wall during testing. Linear potentiometers were positioned to record the total wall deflection, anchorage displacements, nail slip between the sheathing and framing, and framing separation at the corners, as detailed in Table 4.9 and Fig. 4.18. In the quasi-static tests, the applied load was recorded by a load cell positioned between the hydraulic jack and the top loading channel, (see Fig. 4.15). Electrical output from the load cell and potentiometers was stored in the Solatron Data Acquisition System [100]. Records of load and deflection were scanned by the Solatron system at every 5 mm total wall deflection $\Delta_T$. Continuous graphic output of applied load $V$ and total wall deflection $\Delta_T$ was monitored separately on an X-Y plotter, and enabled the performance of the wall to be assessed throughout the testing procedure.

Table 4.9 Displacement Measurements for Test Walls.

<table>
<thead>
<tr>
<th>SERIES</th>
<th>TEST WALLS</th>
<th>PURPOSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>all</td>
<td>Measurement of the total wall deflection</td>
</tr>
<tr>
<td>B</td>
<td>all</td>
<td>Measurement of anchorage connections deflection</td>
</tr>
<tr>
<td>C</td>
<td>S1 to S4</td>
<td>Monitor sheathing nail slip parallel to sheathing edge</td>
</tr>
<tr>
<td>D</td>
<td>S1 to S4</td>
<td>Monitor sheathing nail slip perpendicular to sheathing edge</td>
</tr>
<tr>
<td>E</td>
<td>S1 to S4</td>
<td>Monitor framing joint movement</td>
</tr>
</tbody>
</table>

In the dynamic tests, load was monitored by the load cell located between the base beam and shaketable (see Fig. 4.15). Acceleration records of the inertia mass, and displacement and acceleration traces of the shaketable were also recorded during dynamic tests. An accelerogram was mounted at the centre of gravity of the inertia mass, and on the shaketable adjacent to the load-cell.
The displacement response of the shaketable was measured by an internal LVDT within the hydraulic actuator connected to the shaketable [99]. Time-history traces of shaketable displacement and acceleration were recorded on a Brush Mark 280 chart recorder. Time-history traces of wall load, acceleration and displacements were recorded on a Bryans Southern Ultraviolet Oscillograph. The time-history traces were later digitized to generate load-deflection hysteresis loops.

**Fig. 4.18** Details of Instrumentation.
4.4.3 Loading Sequence

4.4.3.1 Quasi-Static Loading

Test Walls S1 to S4 were subjected to (displacement controlled) reverse cyclic loads to successive total wall displacement $\Delta_T$ peaks of 15, 30, 45 mm etc, or until failure. Two loading cycles were completed to each displacement peak as illustrated in Fig. 4.19. The incremental loading sequence shown in Fig. 4.19 enabled the degradation in wall stiffness to be monitored throughout the wall displacement range until failure.

The loading sequence illustrated in Fig. 4.19 is similar to the displacement ductility criterion commonly used at the University of Canterbury for reinforced concrete sub-assemblies [103]. It was not possible however to apply the ductility criterion directly to the plywood sheathed test walls because their load-deflection response does not exhibit a yield displacement
necessary for the definition of ductility. Rather, for these test walls a nominal yield displacement \( \Delta_y \) was used. This was estimated as the predicted wall displacement occurring at the wall design earthquake load \( V_E = 1.25 V_{ws} \), where \( V_{ws} \) is the permissible wind/seismic load (see Chapter 2). The nominal yield displacement was estimated to be \( \Delta_y = 15 \text{ mm} \) for all test walls using the Table 2.1 deflection equation, taking the average nail slip \( \Delta_{av} \) at the design earthquake load \( V_E \) as 1.5 mm (extrapolated from NZS 3603: Fig. 20), and expecting 2-3 mm wall deflection from anchorage displacements. The loading sequence therefore approximates the loading sequence of two cycles to displacement ductilities \( \mu_A \) of 1, 2, 4, 6, etc, permitting the wall performance to be compared with other seismic resisting assemblies.

Fig. 4.20  Variation of dynamic magnification factor with frequency ratio and damping ratio.
Fig. 4.21 Typical shakeable acceleration-history traces for sinusoidal motion.

Table 4.10 Shakeable Sinusoidal Loading Sequence

<table>
<thead>
<tr>
<th>No. cycles x amplitude</th>
<th>RUN</th>
</tr>
</thead>
<tbody>
<tr>
<td>TEST WALL 1</td>
<td>1</td>
</tr>
<tr>
<td>SS5</td>
<td>5 x 15</td>
</tr>
<tr>
<td>SS6</td>
<td>5 x 15</td>
</tr>
<tr>
<td>SS7</td>
<td>5 x 15</td>
</tr>
<tr>
<td>SS8</td>
<td>5 x 15</td>
</tr>
</tbody>
</table>
4.4.3.2 Sinusoidal Shake Table Loading

Test walls S5 and S8 were subjected to sinusoidal shake table loading having excitation frequency of 1.5 Hz. The tests were carried out in a sequence of gradually increasing shake table displacement amplitude of 15, 22.5, 30, 37.5, 45 mm etc; or until failure. Generally, five cycles of sinusoidal shake table motion were used for each amplitude during the initial stages of the loading sequence, and then only two cycles for the remaining stages. There were some departures from this sequence, and full details of the test sequence for each test wall are shown in Table 4.10. Theoretical shake table accelerations are shown in Table 4.11. Figure 4.20 plots dynamic magnification factor $D$ with damping and frequency ratio for a single degree of freedom oscillator subjected to harmonic ground motion. The shake table excitation frequency was chosen to be approximately one half the natural frequency of the specimen, so as to produce a dynamic magnification factor of about 0.5 during the initial stages of the test (see Fig. 4.20). The natural frequency of Test Wall S5, (based on the theoretical tangent stiffness at the wall design earthquake load $V_E$) was 3.5 Hz. As the shake table amplitudes increased, the wall stiffness decreased and the test wall natural frequency tended towards the excitation frequency. This resulted in significantly increased displacement demands on the wall, leading eventually to wall failure. Figure 4.21 reproduces some typical acceleration-time traces recorded by the accelogram mounted on the deck of the shake table. The high frequency component in the traces were believed to be related to the characteristics of the shake table facility, but this was unlikely to have influenced the response of the wall. Each recorded acceleration trace was digitized, and used in the analytical modelling of the test walls response, reported in the following chapter.
Table 4.11  Shaketable Amplitudes and Accelerations.

<table>
<thead>
<tr>
<th>RUN</th>
<th>Selected Nominal Shaketable Amplitude $A_g$ (mm)</th>
<th>Calculated Shaketable Maximum Acceleration* (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15</td>
<td>0.14</td>
</tr>
<tr>
<td>2</td>
<td>22.5</td>
<td>0.20</td>
</tr>
<tr>
<td>3</td>
<td>30</td>
<td>0.27</td>
</tr>
<tr>
<td>4</td>
<td>37.5</td>
<td>0.34</td>
</tr>
<tr>
<td>5</td>
<td>45</td>
<td>0.41</td>
</tr>
<tr>
<td>6</td>
<td>52.5</td>
<td>0.48</td>
</tr>
<tr>
<td>7</td>
<td>60</td>
<td>0.54</td>
</tr>
<tr>
<td>8</td>
<td>67.5</td>
<td>0.61</td>
</tr>
</tbody>
</table>

* $A_g(2\pi f)^2$  $f$ = excitation frequency = 1.5 Hz

4.4.3.3 Simulated Earthquake Shaketable Loading

Test walls S9 to S11 were subjected to the May 1940 El Centro NS component earthquake ground motion record. The time axis of the acceleration record was not scaled. The walls were subjected to repeated earthquake attacks until failure was imminent. After the first test of each wall, the displacement component of the record was increased by 20%. The El Centro record was adopted for this study because of its long association with analytical and experimental studies of earthquake resistant structures.

A typical acceleration and displacement versus time trace of the shaketable is reproduced in Fig. 4.22.
Fig. 4.22 Typical shakeable displacement and acceleration traces for the first 20 seconds of the El Centro 1940 NS ground record.
5

EXPERIMENTALLY OBSERVED BEHAVIOUR OF TEST WALLS

5.1 INTRODUCTION

This chapter contains the results of tests conducted on the eleven full scale plywood sheathed shear walls, (walls detailed in Table 4.4). The results from the resonant tests, quasi-static tests, sinusoidal shaketable, and El Centro 1940 shaketable tests are described in turn. The chapter concludes with design recommendations, and a summary of observed experimental behaviour.

5.2 RESULTS FROM RESONANT TESTING OF TEST WALLS S1 AND S4

Prior to subjecting Test Walls S1 and S4 to quasi-static reverse cyclic loads, each was subjected to small amplitude sinusoidal shaketable testing in order to establish their elastic damping characteristics. The damping characteristics were evaluated from the observed steady state harmonic response, and free vibration decay.

The test walls were subjected to 10 cycles of sinusoidal shaketable motion at each specified amplitude and frequency. The
shaketable frequency was adjusted to give a range of excitation frequencies either side of the natural frequency of the test wall. The shaketable excitation amplitude $A_g$ was 1 mm and 2 mm (nominal) for Test Wall S1 and S4 respectively.

![Graph showing wall deflection over time](image)

**Fig. 5.1 Wall Total Deflection versus Time Response Trace.**

Figure 5.1 shows a typical displacement-history response trace. The steady state displacement amplitude $A$ was easily distinguishable from the response trace. The free vibration response trace was recorded after each harmonic excitation (see Fig. 5.1), and was typical of underdamped systems whose response is dominated by a single vibration mode. The low mass of the test wall itself compared to the inertia mass is a feature of the testing arrangement shown in Fig. 4.15, and this caused the response of the test wall to be dominated by the first mode of vibration.

A frequency response curve plotting dynamic magnification factor $D = A/A_g$ against shaketable frequency is presented in Fig. 5.2, where $A$ is the steady state response amplitude and $A_g$ is the shaketable excitation amplitude. The response curves indicate a fundamental natural frequency of 3.3 and 3.4 Hz for Test Walls S1 and S4 respectively.
Fig. 5.2 Frequency Response Curves for Wall S1 and S4.

The elastic damping ratio $\lambda$ of the test walls was determined, with reference to Fig. 5.1 and 5.2 for notation, using three methods:

1. Free vibration decay (logarithmic decrement),
   \[ \lambda = \frac{1}{2\pi} \ln\left(\frac{V_1}{V_2}\right) \]

2. Resonant amplification
   \[ \lambda = \frac{1}{2D_{\text{max}}} \]
   and

3. Bandwidth
   \[ \lambda = \frac{(f_2 - f_1)}{(f_2 + f_1)} \]

The method of evaluating the damping ratio $\lambda$ for the bandwidth method is illustrated by with the frequency response curve for Wall S4, shown in Fig. 5.2. A horizontal line has been constructed across the curve at $1/\sqrt{2}$ times the resonant-response value $D_{\text{max}}$. The difference between the frequencies at which this line intersects with the response curve, and the resonant frequency is denoted as $f_1$ and $f_2$. The theory for each method is derived elsewhere [95].
Table 5.1 tabulates the elastic damping ratio $\lambda$ obtained by the above methods for Test Walls S1 and S4. The damping ratio $\lambda$ obtained from the free vibration decay method is the average value of all frequency tests for that test wall.

<table>
<thead>
<tr>
<th>METHOD</th>
<th>WALL S1</th>
<th>WALL S4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free vibration decay</td>
<td>8.2%</td>
<td>18.7%</td>
</tr>
<tr>
<td>Resonant Amplification</td>
<td>7.0%</td>
<td>13.0%</td>
</tr>
<tr>
<td>Bandwidth Method</td>
<td>14.0%</td>
<td>20.0%</td>
</tr>
</tbody>
</table>

Damping values obtained using the bandwidth method are significantly higher than those values obtained using the other two methods. It is evident from Fig. 5.2 that the bandwidth method requires accurate plotting of the response curve at resonance, and within the bandwidth regions. It is believed that compared to the other two methods the higher damping values predicted by this method are due to the wall amplitude not being measured at $D_{\text{max}}/2$. Consequently, the frequencies $f_1$ and $f_2$ at the intercept of the response curve with the horizontal line $D_{\text{max}}/2$ are approximate only. Neglecting the results from the bandwidth method, the average elastic damping ratios for Test Walls S1 and S4 were 8% and 16% respectively.

The significantly higher damping values of Test Wall S4 (tested immediately after fabrication) compared to Test Wall S1 (tested 4 months after fabrication) appear to be due to higher friction forces at the contact surface between the sheathing and framing. These higher friction forces are due to the sheathing nails of Test Wall S4 applying greater clamping pressure between the plywood and framing than the sheathing nails in Test Wall S1. This was because the initial nail tension that develops with nail driving reduces with the age of the nailed joint, (see Section 3.2).
Fig. 5.3 Wall Load versus Total Deflection Hysteresis Loops for Walls S1 to S4.
Fig. 5.3 Continued.
5.3 RESULTS AND DISCUSSION OF TEST WALLS S1 TO S4 WHICH WERE SUBJECTED TO QUASI-STATIC REVERSE CYCLIC LOADING

5.3.1 General Overall Behaviour

Figure 5.3 shows the recorded load versus total deflection (see Fig. 2.3. for notation) hysteresis loops for Test Wall S1 to S4. Initial cycling to the total wall deflection of 15 mm ($\Delta_T = 15$ mm) resulted in minimal degradation. The design earthquake load $V_E$ of 23 kN for Test Walls S1, S3 and S4 and 11.5 kN for Test Wall S2 was developed at a displacement of approximately $\Delta_T = 15$ mm in the case of Test Walls S1 to S3 and approximately $\Delta_T = 11$ mm for Test Wall S4, where $V_E = 1.25 V_{WS}$ and $V_{WS}$ is the NZS 3603:Table 29 [11] permissible wind/seismic load.

Rigid body rotations arising from vertical anchorage displacements (Fig. 4.18:gauges B1 and B2) contributed to between 20% and 25% of the total wall deflection $\Delta_T$ at the design earthquake load $V_E$. Movement of the shear anchorage connections (Fig. 4.18:gauges B3 and B4) was less than 1% of the total wall deflection. Wall deflection arising from anchorage displacements (gauges B1 to B4) have been subtracted from the total wall deflection $\Delta_T$ to produce the load versus lozenging deflection hysteresis loops presented in Fig. 5.4.

Parent load-deflection curves constructed from the envelopes of the total and lozenging hysteresis loops for Test Walls S1 and S4 shown in Fig. 5.3 and 5.4) are plotted in Fig. 5.5. Applied load is specified here in terms of the NZS 3603:1981 permissible wind/seismic load $V_{WS}$.

When loading Test Wall S1 in the initial loading direction to a total wall deflection of 35 mm ($\Delta_T = + 35$ mm), the top framing plate was observed to buckle laterally out of plane at each end of the wall, as shown in Plate 5.1. The lateral movement between
Fig. 5.4 Wall Load versus Lozenging Deflection Hysteresis Loops for Walls S1 to S4.
Fig. 5.4 Continued.
Plate 5.1 WALL S1: Out of Plane Buckling of top framing plate.

Plate 5.2 WALL S1: Buckling of Plywood.
the top framing plate and the joists restricted vertical slip of
the plywood along the west framing stud, and this in turn caused
the sheathing nail slip to increase along the centre framing stud
at the expense of sheathing nail slip along the west framing
stud. The wall was unloaded at $\Delta_T = +35\text{mm}$ and the connection
between the top framing plate and joist was strengthened with
four 100 x 4.0 mm nails at the top corner of the wall. (The
framing plate to joist connection was also strengthened in all
subsequent walls prior to testing). Because of this buckling,
the loading sequence of Fig. 4.19 was departed from during the
testing of Wall S1, and one displacement cycle was made to each
of $\Delta_T = +35\text{mm}, -45\text{mm}, +45\text{mm}$ and $-45\text{mm}$ etc. as shown in
Fig. 5.3.

The ultimate strength of the wall was developed at a lozenging
deflection ($\Delta_L$) of between 30 and 45 mm. The ultimate strength
itself was approximately $1.4 V_E (1.8 V_{WS})$ for Walls S1 to S3 and
$1.9 V_E (2.4 V_{WS})$ for Wall S4. At ultimate load, the plywood of
Walls S1 and S3 buckled as shown in Plate 5.2. The buckling was
constrained by the interior framing studs and appeared not to
affect wall strength.

Wall S4 was stronger than Walls S1 and S3. This was due to the
strength of the sheathing nailing of Walls S1 and S3 reducing
with joint age, as described in Chapter 3 (Walls S1 and S3 were
several months older than Wall S4, as reported in Chapter 4).

The ultimate strength of the test walls in the initial loading
direction was generally higher than that in the opposite loading
direction. For Walls S2 to S4, the initial loading direction
corresponds to the $\Delta_T = +45\text{mm}$ loading cycles (Fig. 5.3 to 5.5).
For Wall S1 the initial loading direction at ultimate load
corresponds to the $\Delta_T = -45\text{mm}$ loading cycle for the reasons
discussed above. The difference in wall strength between
positive and negative loading directions is characteristic of
nailed joint behaviour, as discussed previously in Chapter 3.
Fig. 5.5 Parent Load-Deflection curves for Walls S1 to S4. Load Expressed in Terms of NZS 3603:1981 Permissible Wind/Seismic Wall Load $V_{WS}$. (a) Load versus Total Deflection. (b) Load versus Lozenging Deflection.
Plate 5.3  WALL S1: Sheathing nail failure.

Plate 5.4  WALL S2: Sheathing nail failure.

Plate 5.5  WALL S3: Buckling of plywood along top framing plate.
5.3.1.1 Failure Modes

Wall S1:

Failure of Wall S1 occurred during the first loading cycle to \( \Delta_T = -75 \) mm when sheathing nails failed along the connection between the centre framing stud and east plywood panel. Approximately 2% of the nails failed from fatigue, with the remainder of the nails either pulling through the sheathing or pulling out of the framing stud, as shown in Plate 5.3. The wall was finally loaded to \( \Delta_T = +90 \) mm, and despite the previous failure of the nails along the connection between the centre framing stud and east plywood panel, a load of almost 20kN (0.9 \( V_e \)) was sustained.

Wall S2:

Failure of Wall S2 occurred during the first loading cycle to \( \Delta_T = +75 \) mm when the sheathing nails failed along the connection between the west framing stud and the plywood. Nail failure was similar to that observed for Wall S1. Plate 5.4 shows the condition of the failed connection at the end of the test.

Wall S3:

Failure of Wall S3 occurred when loading the wall to \( \Delta_T = +75 \) mm. The nails pulled out from the top framing plate along the connection between the west plywood panel and the framing. Plate 5.5 shows the condition of connection at \( \Delta_T = +75 \) mm.

Wall S4:

Failure of Wall S4 occurred when the nails pulled out of the framing during the load cycle to \( \Delta_T = +90 \) mm. Plate 5.6 gives a view of the connection between the framing stud and the plywood sheathing at the end of the test. The sheathing nails were observed to progressively withdraw during cycling, as reported in the nailed joint tests incorporating 12 mm thick plywood (see Chapter 3).
Plate 5.6 WALL S4: at the end of test.

Plate 5.7 Deformed shape of nails from Walls S1 to S4 after end of test.
At completion of testing, some nails were carefully removed from Walls S3 and S4, and were taken as representative of the nails in 7.5 mm and 12 mm thick plywood. The deformed shapes of the nails are shown in Plate 5.7. Generally, the nails from the 7.5 mm thick plywood were deformed in single curvature and the nails from the 12 mm thick plywood were deformed in double curvature. The 12 mm thick plywood, however, was not of sufficient thickness to enforce full nail head fixity.

5.3.1.2 Overall Displacement Capacity

The load carrying capacity of Walls S1 to S3 was sustained to lozenging deflections $\Delta_i$ of between 40 and 50 mm. At larger lozenging deflections there was a gradual reduction in strength, and by 75 mm lozenging deflection the load carrying capacity had reduced 45% to 0.8 $V_E$. In contrast, the load-deflection response (Fig. 5.5) of Wall S4 indicates a more stable parent curve up to lozenging deflections of 75 mm or more. The more stable parent curve of Wall S4 is attributed to the improved load-slip characteristics of the 50 x 2.89 mm nails driven through 12 mm thick plywood compared to 7.5 mm thick plywood, as reported in Chapter 3.

Table 5.2 Displacement Ductility Capacity $\mu_\Delta$.

<table>
<thead>
<tr>
<th>Wall</th>
<th>$\Delta_y$</th>
<th>$\Delta_{\text{max}}$</th>
<th>$\mu_\Delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>15</td>
<td>75</td>
<td>5</td>
</tr>
<tr>
<td>S2</td>
<td>15</td>
<td>75</td>
<td>5</td>
</tr>
<tr>
<td>S3</td>
<td>15</td>
<td>75</td>
<td>5</td>
</tr>
<tr>
<td>S4</td>
<td>11</td>
<td>90</td>
<td>8</td>
</tr>
</tbody>
</table>

Table 5.2 tabulates the recorded nominal yield displacement $\Delta_y$, maximum displacement $\Delta_{\text{max}}$ and the displacement ductility capacity $\mu_\Delta = \Delta_y/\Delta_{\text{max}}$ for Walls S1 to S4, where the yield displacement $\Delta_y$ is defined here as the wall deflection at the design earthquake load $V_E$. Walls S1 to S3 (7.5 mm thick plywood) exhibited displacement ductility capacity in excess of $\mu_\Delta = 5.0$, and Wall S4 exhibited a value of $\mu_\Delta = 8.0$. 
5.3.1.3 Hysteretic Response

The load-deflection hysteresis loops shown in Fig. 5.3 and 5.4 are similar to the characteristic pinched loops observed in the nailed joint tests described in Chapter 3. The pinching is characteristic of nailed joints, as explained in Chapter 3.

Fig. 5.6 Wall Load versus Vertical Anchorage Deflection Hysteresis Loops for Wall S3.
5.3.2 **Anchorage Connection Behaviour**

Figure 5.6 shows the applied load vs vertical deflection hysteresis loops of the east and west vertical anchorage connections of Wall S3. The Fig. 5.6 hysteresis loops are typical of the behaviour of the vertical anchorage connections in all four walls, and show that the connections remained essentially elastic exhibiting little stiffness degradation.

Parent load-deflection curves constructed from the recorded applied load versus vertical deflection hysteretic loops of the vertical anchorage connections of Walls S1, S3 and S4 are compared in Fig. 5.7. The nail plate and tie rod assembly (see Fig. 4.10 (b)) exhibited high stiffness up to displacements of about 3 mm. Less than 10\% of this was attributed to elongation of the 12 mm dia. ties. The much larger displacements observed at the east hold-down connection of Wall S4 are believed to be due to one of the two 12 mm dia. ties being loose prior to testing. At an applied load of about 30 kN, yielding of the other 12 mm dia. tie occurred, and continued until the slackness of about 9 mm in the loose tie was taken up. Thereafter, load was shared by both ties and further displacements were less than 1 mm.

Compressive thrust was transferred to the reaction frame through perpendicular-to-grain compression at the interface between the framing stud and the bottom framing plate (Fig. 4.10). At the design earthquake load $V_E$, deflection of the anchorage connection was typically 1 mm in tension and 2 mm in compression. Compressive stresses of 3 MPa were sustained without permanent perpendicular-to-grain crushing of the timber, but thereafter some crushing and stiffness degradation of the connection was observed, as seen in Fig. 5.7. Excessive cupping or twisting at the east end of the bottom framing plate, is believed to have caused the significantly reduced stiffness in compression of the east vertical connection of Wall S1 compared to the other connections (see Fig. 5.7).
Fig. 5.7 Parent Wall Load versus Vertical Anchorage Deflection curves for Walls S1, S3 and S4.

5.3.3 Framing Joint Behaviour

The rotation of the plywood over the framing induced nail force components acting in the direction perpendicular to the framing. This in turn caused the framing joint forces X and Y, for which predictions were developed in Chapter 2. The NZS 3603:1981 [11] permissible wind/seismic load and the estimated strength of the framing joint connections of Walls S1 to S4 are summarised in Table 5.3.

The framing joint forces exceeded the withdrawal resistance and shear strength of the two 100 x 4.0 mm nails connecting the framing members of Walls S1 and S2, and up to 8 mm joint separation was recorded. The vertical and horizontal movement between the top framing plate and the east framing stud, and between the top framing plate and the centre framing stud of Walls S1 and S2 is plotted in Fig. 5.8 against total wall deflection \( \Delta_w \). Framing joint separation was greatest when the rotation of the plywood tended to prise the edge framing joints open.
Table 5.3 Permissible Wind-Seismic Load and Strength of Framing Connections.

<table>
<thead>
<tr>
<th></th>
<th>TEST WALLS 1, 2 &amp; 5 - 11</th>
<th>TEST WALL 3</th>
<th>TEST WALL 4</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>KN</strong></td>
<td>two 100 x 4.0 mm nails</td>
<td>two 100 x 4.0 mm nails plus one &quot;multigrip&quot; connector</td>
<td>two 100 x 4.0 mm nails plus two &quot;multigrip&quot; connectors</td>
</tr>
<tr>
<td><strong>Y_1</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permissible Wind/Seismic</td>
<td>0.9(1)</td>
<td>2.6(3)</td>
<td>4.3(3)</td>
</tr>
<tr>
<td>Expected Strength</td>
<td>1.4(2)</td>
<td>7.9(3)</td>
<td>14.4(3)</td>
</tr>
<tr>
<td><strong>Y_2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permissible Wind/Seismic</td>
<td>40.0(4)</td>
<td>40.0(4)</td>
<td>40.0(4)</td>
</tr>
<tr>
<td>Expected Strength</td>
<td>57.0(5)</td>
<td>57.0(5)</td>
<td>57.0(5)</td>
</tr>
<tr>
<td><strong>X_1</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permissible Wind/Seismic</td>
<td>0.7(1)</td>
<td>1.4(3)</td>
<td>2.1(3)</td>
</tr>
<tr>
<td>Expected Strength</td>
<td>1.9(6)</td>
<td>8.0(3)</td>
<td>14.1(3)</td>
</tr>
<tr>
<td><strong>X_2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permissible Wind/Seismic</td>
<td>0.7(1)</td>
<td>1.4(3)</td>
<td>2.1(3)</td>
</tr>
<tr>
<td>Expected Strength</td>
<td>1.9(6)</td>
<td>4.3(3)</td>
<td>6.7(3)</td>
</tr>
</tbody>
</table>

(1) NZS3606:1981 permissible wind/seismic load
(2) Ultimate capacity = 2.5 times NZS3603:1981 basic load \( P_w \)
(3) Wind/seismic load from manufacturers technical publication [97]
(4) NZS3603:1981 \( K_3 = 1.33 \)
(5) Ultimate stress = 2.2 times NZS3606:1981 basic stress \( f_p \)
(6) Ultimate capacity = 4.15 times NZS3603:1981 basic load \( P'_w \).
Fig. 5.8 Separation between Framing Members of Walls S1 and S2.

Closing joint force Y (vertical) of the top framing joints was resisted by perpendicular-to-grain compression at the interface between the framing stud and framing plate. Consequently, closing vertical displacements of the framing joints were
generally less than 1 mm (Fig. 5.8). The closing vertical displacement of the top corner framing joint connection of Wall S2 was 3 mm. However, the reason for the significantly greater displacement for this connection could not be found.

The vertical separation of the top centre framing joint connection in Wall S1 was 4 mm (Fig. 5.8), despite zero vertical joint force being predicted there (Table 2.10). The vertical separation appeared to be due to the framing plate and joists being lifted as a complete assembly at the edge framing joint, as shown in Fig. 5.9(a). Such separation is not accounted for in the proposed theory (see Chapter 2).

Because of the large joint separation observed during the testing of Walls S1 and S2, the framing joint connections of Walls S3 and S4 (except the two bottom corner joints) were strengthened with Lumberlock "multigrip" [97]. One multigrip connector, fastened to the edge opposite the sheathing, was used to strengthen the joints of Wall S3. Two multigrip connectors, one on each framing edge, were used to strengthen the joints of Wall S4. The multigrip connector fastened to the sheathing edge of the framing of Wall S4 was recessed into the framing so that it did not restrict the rotation of the sheathing over the framing. The permissible wind/seismic loads for the strengthened framing joint connections of Walls S3 and S4 are summarised in Table 5.3. The permissible wind/seismic load for the strengthened framing joint connection of Wall S4 is about 3 times that of the unstrengthened joints of Walls S1 and S2.

The single multigrip connector fastened to the framing edge opposite the sheathing in Wall S3, restrained vertical separation to less than 2 mm. However, twisting of the framing stud about its unsheathed edge (see Fig. 5.9(b)), permitted the horizontal separation at the sheathing edge to exceed 8 mm.

The two multigrip connectors used in Wall S4 performed very well, and restricted joint separation to less than 3 mm in both horizontal and vertical directions.
Fig. 5.9 Separation between Framing Members. (a) Between Top Framing Plate and Interior Framing Stud. (b) Between Top Framing Plate and Edge Framing Stud.
5.3.4. Sheathing Nail Behaviour

The measured in-plane rotation of the plywood over the framing members verified that nail slip occurs in directions parallel and perpendicular to the plywood edges. The recorded nail slip components in each direction are presented here separately.

5.3.4.1 Nail Slip Parallel to the Plywood Edges

Table 5.4 tabulates for each wall the recorded nail slip (Fig. 4.18: gauges C) parallel to the plywood edges at each cycle amplitude. The Table 5.4 results for Wall S1 and S4 are shown graphically in Fig. 5.10. Nail slip around the edges of each plywood panel of Wall S1 was uniform up to total wall deflections of $\Delta_T = 30$ mm. Thereafter, there was a tendency for nail slip along only one edge to increase, with little further increase in slip along the other edges (Fig. 5.10). This was also observed in Walls S2 and S3, (see Table 5.1). This occurred because as the incremental stiffness in one nail line became less than that of the other nail lines, nail slip increased there at the expense of the other lines.

In contrast, Wall S4 exhibited nearly uniform nail slip around the edges of each plywood panel up to wall deflections $\Delta_T = 75$ mm (see Fig. 5.10). This improved behaviour is attributed to the improved load-slip characteristics of nailed plywood joints incorporating the thicker 12 mm thick plywood (as discussed in Chapter 3 [see Fig. 3.22]), and also to the stiffer framing joint connections of Wall S4.
### Table 5.4(a) Nail Slip Parallel to Framing Members - Wall S1.

<table>
<thead>
<tr>
<th>Cyclic Peak LOAD (kN)</th>
<th>EAST PANEL</th>
<th>WEST PANEL</th>
<th>Average ( \sigma_{KN} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bottom Plate</td>
<td>East Stud</td>
<td>Top Plate</td>
</tr>
<tr>
<td>15</td>
<td>18.22</td>
<td>0.3</td>
<td>0.4</td>
</tr>
<tr>
<td>-15</td>
<td>20.67</td>
<td>0.8</td>
<td>1.0</td>
</tr>
<tr>
<td>30</td>
<td>26.62</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>-30</td>
<td>30.09</td>
<td>3.5</td>
<td>5.7</td>
</tr>
<tr>
<td>45</td>
<td>40.62</td>
<td>0.4</td>
<td>0.7</td>
</tr>
<tr>
<td>-45</td>
<td>50.88</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>60</td>
<td>27.20</td>
<td>5.0</td>
<td>6.9</td>
</tr>
<tr>
<td>-60</td>
<td>21.86</td>
<td>5.0</td>
<td>6.9</td>
</tr>
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</table>

### Table 5.4(b) Nail Slip Parallel to Framing Members - Wall S2.

<table>
<thead>
<tr>
<th>Cyclic Peak LOAD (kN)</th>
<th>EAST PANEL</th>
<th>WEST PANEL</th>
<th>Average ( \sigma_{KN} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bottom Plate</td>
<td>East Stud</td>
<td>Top Plate</td>
</tr>
<tr>
<td>15</td>
<td>11.65</td>
<td>1.3</td>
<td>1.2</td>
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<tr>
<td>-15</td>
<td>13.0</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>30</td>
<td>16.2</td>
<td>2.4</td>
<td>1.2</td>
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<td>-30</td>
<td>15.2</td>
<td>3.2</td>
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<td>6.0</td>
<td>9.6</td>
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<tr>
<td>75</td>
<td>10.0</td>
<td>6.1</td>
<td>12.8</td>
</tr>
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</table>

### Table 5.4(c) Nail Slip Parallel to Framing Members - Wall S3.

<table>
<thead>
<tr>
<th>Cyclic Peak LOAD (kN)</th>
<th>EAST PANEL</th>
<th>WEST PANEL</th>
<th>Average ( \sigma_{KN} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bottom Plate</td>
<td>East Stud</td>
<td>Top Plate</td>
</tr>
<tr>
<td>15</td>
<td>20.20</td>
<td>0.9</td>
<td>0.4</td>
</tr>
<tr>
<td>-15</td>
<td>22.56</td>
<td>1.6</td>
<td>1.2</td>
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<tr>
<td>30</td>
<td>30.72</td>
<td>2.2</td>
<td>1.3</td>
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<tr>
<td>-30</td>
<td>29.73</td>
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</tr>
<tr>
<td>-45</td>
<td>31.64</td>
<td>6.0</td>
<td>5.0</td>
</tr>
<tr>
<td>60</td>
<td>32.21</td>
<td>4.8</td>
<td>4.8</td>
</tr>
<tr>
<td>-60</td>
<td>28.83</td>
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<td>7.0</td>
</tr>
<tr>
<td>75</td>
<td>23.62</td>
<td>4.9</td>
<td>5.9</td>
</tr>
</tbody>
</table>

### Table 5.4(d) Nail Slip Parallel to Framing Members - Wall S4.

<table>
<thead>
<tr>
<th>Cyclic Peak LOAD (kN)</th>
<th>EAST PANEL</th>
<th>WEST PANEL</th>
<th>Average ( \sigma_{KN} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bottom Plate</td>
<td>East Stud</td>
<td>Top Plate</td>
</tr>
<tr>
<td>15</td>
<td>31.0</td>
<td>1.3</td>
<td>0.5</td>
</tr>
<tr>
<td>-15</td>
<td>26.5</td>
<td>0.9</td>
<td>0.8</td>
</tr>
<tr>
<td>30</td>
<td>39.9</td>
<td>3.2</td>
<td>1.7</td>
</tr>
<tr>
<td>-30</td>
<td>32.5</td>
<td>2.4</td>
<td>2.3</td>
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<td>45</td>
<td>43.8</td>
<td>5.0</td>
<td>3.5</td>
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<tr>
<td>60</td>
<td>42.4</td>
<td>6.5</td>
<td>5.4</td>
</tr>
<tr>
<td>-60</td>
<td>38.2</td>
<td>5.8</td>
<td>6.5</td>
</tr>
<tr>
<td>75</td>
<td>40.9</td>
<td>7.6</td>
<td>7.6</td>
</tr>
</tbody>
</table>
Fig. 5.10 Nail Slip Parallel to Framing Members of Walls S1 and S4.

5.3.4.2 Nail Slip Perpendicular to the Framing Stud

Nail slip in the direction perpendicular to the framing stud at each cycle amplitude is plotted in Fig. 5.11 against framing stud height for Walls S2 to S4. Little nail slip occurred along the middle 1.5 m length of the framing stud. As previously described in Chapter 2, the nail force components acting in the direction perpendicular to the framing stud cause the framing stud to bend, and this in turn reduces the nail slip from the linear nail slip distribution along the length of the framing stud, which would occur if the framing studs were rigid. Consistent with the shear wall elastic theory, Fig. 5.11 shows that the nail slip perpendicular to the interior framing stud members (with their two nail lines) exhibit less nail slip than the edge framing stud members.
Fig. 5.11 Nail Slip Perpendicular to Framing Stud Members of Walls S2 to S4.
5.3.5 Comparison with Proposed Shear Wall Theory

In this section, the results from the test walls just presented are compared with predictions made by the elastic and ultimate strength theories formulated in Chapter 2.

5.3.5.1 Elastic Theory

Wall Stiffness

The load-deflection response of the test walls is predicted in this section using the load-slip characteristics of the nailed sheathing joint tests (see Fig. 4.12). The Fig. 4.12 load-slip curve enables the average nail slip $\Delta_{av}$ to be determined for calculated average nail force $P_{av}$ values (Eq. 2.45), as described in Chapter 2. Values for the modulus of rigidity of the plywood sheathing of $G = 846$ MPa (Table 4.6), and the modulus of elasticity of the framing member of $E = 8000$ MPa (NZS 3603:Table 2) were used in the theoretical prediction (Table 2.1).

The experimental parent lozenging load-deflection response (from Fig. 5.5) and the theoretical prediction given by the proposed theory are compared in Fig. 5.12 for Walls S1 to S3. The theoretical load-deflection response of Wall S4 could not be predicted because no matching nailed sheathing joint tests were available. The theory slightly overpredicts the experimental wall stiffness, and this is believed to be due to the errors made in recording the average nail slip $\Delta_{av}$ during the nailed sheathing joint tests, as discussed previously in Section 4.3.
Fig. 5.12 Comparison of Results from Test Walls S1 to S3 with those from the Proposed Elastic Shear Wall Theory based on the Load-Slip Characteristics of Separate Nailed Joint Tests.
Wall Deflection Arising From Nail Slip Parallel to Sheathing Edge:

The shear wall theory advanced in Chapter 2 (Eq. 2.43) gives the shear wall lozenging deflection $\Delta_{ln}$ arising from nail slip as $\Delta_{ln} = 6 \Delta_{av}$ for sheathing panels having an aspect ratio $\alpha = H/B = 2$, where $\Delta_{av}$ is the average nail slip parallel to the panel edges. Based on the measured load $V$ versus average nail slip values of Table 5.4, and the same values of $G$ and $E$ as in the previous section, theoretical load vs lozenging deflection curves (see Table 2.1) were calculated, and are compared with the experimental points in Fig. 5.13. Excellent agreement between theory and experimental points is obtained.

![Graphs showing load versus lozenging deflection for different walls](image)

Fig. 5.13 Comparison of Results from Test Walls S1 to S4 with those from the Proposed Elastic Shear Wall Theory based on Measured Average Nail Slip parallel to the Sheathing Edge.
Fig. 5.14 Comparison of Results from Measured Nail Slip Perpendicular to Framing Stud to those from the Proposed Elastic Shear Wall Theory.

Nail Slip Perpendicular to Sheathing Edge:

The predicted nail slip in the direction perpendicular to the sheathing edge (Eq. 2.4) is compared with experimental values (measured by gauges D: Fig. 4.18, and as plotted in Fig. 5.11). Theoretical nail slip curves were based on framing modulus of
elasticity of $E = 8000$ MPa, and initial sheathing nail stiffness $k = 650$ N/mm (from Fig. 4.12). The horizontal nail slip $\Delta'_{x}$ at the corners of the sheathing panel was taken as that measured by the appropriate gauge D1, D7, D8 or D14 (see Fig. 4.18). The theoretical curves and experimental points are compared in Fig. 5.14 for Wall S2 and S3. Generally, very good agreement exists between theory and experiment. The discrepancies between theory and experiment when the edge framing stud is subjected to tension is due to the influence of the hold-down connection on the deformed shape of the framing stud. The single hold-down connection positioned on the outside face of the edge framing stud of Wall S2, and the two hold-down connections on each face of the edge framing stud of Wall S3 induce moment at the bottom end of the framing stud, whereas the theory assumes that the framing is pin-jointed at both ends.

<table>
<thead>
<tr>
<th>Table 5.5 Framing Joint Forces at Permissible Wind/Seismic Load $V_{WS}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(KN)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>WALL S1, S3 &amp; S4</td>
</tr>
<tr>
<td>WALL S2</td>
</tr>
</tbody>
</table>

**Framing Joint Forces:**

The joint forces $X$ and $Y$ predicted by the elastic shear wall theory at the NZS 3603:1981 [11] permissible wind/seismic load $V_{WS}$ are tabulated in Table 5.5, together with the elastic stiffness parameter $K_e$, and framing joint force reduction factor $\phi_{x}$. The predicted joint forces exceed the NZS 3603:1981 [11] permissible wind/seismic load of the two 100 x 4.0 nails used as the framing connections in Walls S1 and S2, see Table 5.3.
Fig. 5.15 Measured Angle $\theta$ of Nail Slip with respect to the Longitudinal Axes of the Framing Members of Wall S1.

5.3.5.2 Ultimate Strength Theory

Nail Force Components:

At the completion of testing Walls S1 and S2, the plywood sheathing was removed from the framing, and the angle $\theta$ that the nail slip made with respect to the longitudinal axes of the framing members was measured. The measured angles $\theta$ for Test Wall S1 are presented in Figure 5.15. The nail force component $P_{\cos \theta}$ acting in the direction parallel to the framing, and the component $P_{\sin \theta}$ acting in the direction perpendicular to the framing were calculated assuming that the nail slip was sufficient to develop the ultimate strength of all the nails around the framing. The ultimate strength $P' = 1.15$ KN of the sheathing nails was obtained from the experimental load-slip curve shown in Fig. 4.12. Figure 5.16 shows the experimental
nail force components acting in the directions parallel and perpendicular to the framing of the east panels of Wall S1 and Wall S2.

Table 5.6 Potentiometers used for Evaluating Nail Slip Ratio \( r_s \) for Framing Members of East Panel.

<table>
<thead>
<tr>
<th>( r_s = \frac{\Delta_y}{\Delta_T} )</th>
<th>Bottom framing Plate ( r_{s,P} )</th>
<th>East framing Stud ( r_{s,E} )</th>
<th>Top framing Plate ( r_{s,P} )</th>
<th>Centre framing Stud ( r_{s,C} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>East</td>
<td>Centre</td>
<td>Top</td>
<td>Bottom</td>
<td>East</td>
</tr>
<tr>
<td>( \Delta_y )</td>
<td>C1</td>
<td>C1</td>
<td>D7</td>
<td>D1</td>
</tr>
<tr>
<td>( \Delta_T )</td>
<td>C2-ES*</td>
<td>C4-ES*</td>
<td>C2</td>
<td>C2</td>
</tr>
</tbody>
</table>

See Fig. 4.18 for position of gauges.

*Two additional potentiometers measuring vertical movement of bottom framing joints for Test Wall S2 only.

The experimental nail force components plotted in Fig. 5.16 for the east panel are compared here with the nail force distributions predicted by the ultimate strength theory advanced in Chapter 2. The nail force distributions were predicted using the measured nail slip ratios \( r_{s,s} \) and \( r_{s,p} \) at each end of the framing members (see Section 2.4.5). Only the nail force components of the east sheathing panel are compared here because the nail slip ratios \( r_{s,s} \) and \( r_{s,p} \) were only measured for this panel (Fig. 4.18). Table 5.6 gives the gauges used to evaluate the nail slip ratios for each framing member within the east panel (see Fig. 4.18 for gauge position). Values of nail slip ratio used in the prediction were the average values recorded at \( \Delta_T = \pm 45 \text{ mm} \) (\( \Delta_y = \pm 35 \text{ mm}, \pm 45 \text{ mm} \) for Wall S1), i.e. the wall deflection corresponding to the ultimate strength of the wall.

Table 5.7 tabulates the average measured nail slip ratios \( r_{s,s} \), \( r_{s,p} \), and the inelastic stiffness parameters \( K_I \) used for the theoretical predictions of Walls S1 and S2. The secant sheathing nail stiffness \( k_s = 115 \text{ N/mm} \), required for determining the inelastic stiffness parameter \( K_I \), and the ultimate strength of the nail \( P' = 1.150 \text{ kN} \) were determined from the Fig. 4.12 sheathing nail load-slip curve. Figure 5.16 compares the predicted nail force components with the experimental points, and shows the very good agreement obtained.
Fig. 5.16 Comparison of Results from Experimental Nail Force Components of Walls S1 and S2 with those from the Proposed Strength Theory.
Fig. 5.16 Continued.
Table 5.7 Measured Nail Slip Ratio \( r_{s,s} \), \( r_{s,p} \).

<table>
<thead>
<tr>
<th>Test Wall</th>
<th>Bottom Framing Plate ( r_{s,p} )</th>
<th>East Framing Stud ( r_{s,s} )</th>
<th>Top Framing Plate ( r_{s,p} )</th>
<th>Centre Framing Stud ( r_{s,s} )</th>
<th>Average</th>
<th>Secant Stiffness</th>
<th>Inelastic Stiffness Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>East Centre Top Bottom East Centre Top Bottom</td>
<td>( r_{s,s} ) ( r_{s,p} )</td>
<td>( r_{s,s} ) ( r_{s,p} )</td>
<td>( r_{s,s} ) ( r_{s,p} )</td>
<td>( k_s )</td>
<td>( K_{i,East} )</td>
<td>( K_{i,Centre} )</td>
</tr>
<tr>
<td>S1</td>
<td>1.27 1.14 2.20 1.94 0.48 2.17 2.50 3.13 2.49 1.33 115</td>
<td>0.023</td>
<td>0.111</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>1.69 1.20 2.52 2.06 0.78 1.18 2.52 4.19 2.82 1.21 115</td>
<td>0.044</td>
<td>0.023</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>- - 1.38 1.21 0.50 0.85 2.12 1.83 1.64 0.68 - - -</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td>- - 0.88 1.04 0.49 0.69 1.13 1.34 1.10 0.59 - - -</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Framing joint separation not measured and assumed zero.

Wall Ultimate Strength:

The experimental nail force components (see Fig. 5.16) are used to predict the ultimate strength of the east panel of Walls S1 and S2. The experimental horizontal framing joint force \( X' \) was evaluated by considering moment equilibrium of the experimental sheathing nail force components about the mid height of the framing stud. The experimental framing joint forces \( X' \) acting at the top of the east and centre framing stud are tabulated in Table 5.8. Also tabulated in Table 5.8, is the summation of the horizontal nail force components \( \int P_x' \) acting in the direction parallel to the framing plate. Horizontal equilibrium of the top framing plate (see Fig. 5.16) requires the strength of the east panel \( V' \) to be \( V' = \int P_x' + X' \) East + X' Centre, and this is tabulated in Table 5.8, along with the recorded experimental strength of the panel, assuming that each panel carries half the recorded load resistance. There is very good agreement between the east panel strength determined by the measured angles and the recorded panel resistance.

Table 5.8 Ultimate Strength of Test Walls.

<table>
<thead>
<tr>
<th>Experimental Wall Strength/Panel (KN)</th>
<th>Theoretical Wall Strength/Panel (KN)</th>
<th>THEORY</th>
<th>EXPERIMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>TEST WALL</td>
<td>Recorded Wall Strength</td>
<td>Wall Strength determined from measured nail slip angle ( \theta )</td>
<td>( r_s )</td>
</tr>
<tr>
<td>S1</td>
<td>16.0 14.4 1.3 0.6 16.3 1.10 1.75 0.76 0.920 0.082 16.9 3.0 19.9 1.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>8.3 7.7 0.6 0.6 8.9 1.04 1.76 0.73 0.922 0.082 8.5 1.5 10.0 1.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>16.5 - - - - 1.00 1.75 0.80 0.944 0.082 17.4 3.0 20.4 1.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td>22 - - - - 1.11 1.43 0.88 0.900 0.095 - - -</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( P' = 1.150 \text{ kN} \)
Table 5.9 Average Framing Joint Slip Ratios.

<table>
<thead>
<tr>
<th>Test Wall</th>
<th>$r_{c,x}$</th>
<th>$r_{c,y}$</th>
<th>$r_{c,ave}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>0.37</td>
<td>0.34</td>
<td>0.35</td>
</tr>
<tr>
<td>S2</td>
<td>0.41</td>
<td>0.31</td>
<td>0.36</td>
</tr>
<tr>
<td>S3</td>
<td>0.43</td>
<td>0.19</td>
<td>0.31</td>
</tr>
<tr>
<td>S4</td>
<td>0.21</td>
<td>0.23</td>
<td>0.22</td>
</tr>
</tbody>
</table>

The experimental ultimate strength values are also compared in Table 5.8 with theoretical predictions. The framing joint slip ratios $r_{c,x}$ and $r_{c,y}$ required for determining the nail slip ratio $r_s$ for the panel (see Fig. 2.24) and hence the wall ultimate strength, are tabulated in Table 5.9. The Table 5.9 slip ratios are the average measured values of all framing joints within the east panel at $\Delta T = \pm 45$ mm ($\Delta T = +35$ mm and -45 mm for Wall S1). Table 5.10 tabulates the predicted nail slip ratio $r_s$ for the east panel (Eq. 2.75), based on the Table 5.9 joint slip ratios, and an assumed inelastic stiffness parameter for the east and centre framing stud members of $K_{I,East} = K_{I,Centre} = 0.01$.

Table 5.10 Nail Slip Ratios $r_s$ for East Panel

<table>
<thead>
<tr>
<th>Test Wall</th>
<th>Experimental $r_s$</th>
<th>Theoretical $r_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>1.55</td>
<td>1.10</td>
</tr>
<tr>
<td>S2</td>
<td>1.70</td>
<td>1.04</td>
</tr>
<tr>
<td>S3</td>
<td>0.95</td>
<td>1.00</td>
</tr>
<tr>
<td>S4</td>
<td>0.80</td>
<td>1.10</td>
</tr>
</tbody>
</table>
Table 5.10 shows reasonably good agreement between the experimental nail slip ratios \( r_s \) (average values for \( \Delta_T = \pm 45 \text{ mm} \) given in Table 5.4) and theoretical values for Walls S3 and S4. The difference in the predicted and experimental nail slip ratio \( r_s \) for walls S1 and S2 is believed to be attributed to the nail forces around the framing not being uniform (see Fig. 5.10), as assumed in the theory.

Table 5.8 tabulates the theoretical nail slip ratios for the panel \( r_{s'} \), framing stud \( r_{s'} \), framing plate \( r_{s'p'} \), the framing plate strength parameter \( \phi'_p \) and the horizontal framing joint strength parameter \( \phi'_x \) for Walls S1 to S4. Also tabulated in Table 5.8 are the summation of the predicted nail force components in the direction parallel to the framing plate \( \sum P'_x = \phi'_p P' B/c; \) the predicted horizontal framing joint force \( X' = \phi'_x P' B/c; \) and the predicted strength of the east panel \( Y' = \sum P'_x + 2X' \) for Walls S1 to S3. The ultimate strength \( P' \) of the sheathing nails within Wall S4 was unknown, and consequently the predicted strength of Wall S4 is not given in Table 5.8. The proposed ultimate strength theory over-predicts the experimental wall strength by 20%.

**Framing Joint Forces:**

The experimental nail force components (see Fig. 5.16) are used to predict the framing joint forces \( X' \) and \( Y' \) corresponding to the ultimate strength of the wall. Table 5.11 tabulates for Walls S1 and S2, the experimental joint forces determined by moment equilibrium of the experimental nail force components about the mid-length of each framing member. The theoretical predictions of the joint forces \( X' \) and \( Y' \) are also tabulated in Table 5.11, based on the assumed inelastic stiffness parameter \( K_{1, \text{East}} = K_{1, \text{Centre}} = 0.01 \), and the predicted nail slip ratios given in Table 5.9. There is reasonably good agreement between experimental and theoretical joint forces.
Table 5.11 Framing Joint Forces.

<table>
<thead>
<tr>
<th>Experimental Joint forces determined from measured</th>
<th>Theoretical Joint forces (assuming $K_{I/g} = K_{l/l} = 0.10$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top Framing Joints</td>
<td>Bottom Framing Joints</td>
</tr>
<tr>
<td>$x'$ East</td>
<td>$y'$ East</td>
</tr>
<tr>
<td>WALL S1 1.3 2.2 1.4 0.4 1.4 2.8 1.4 0.0 1.35 2.50 1.40 0.20 1.51 2.02 3.02 0.00</td>
<td></td>
</tr>
<tr>
<td>WALL S2 0.6 2.0 1.0 0.3 0.8 2.3 1.1 0.4 0.70 2.15 1.05 0.35 0.75 0.97 1.51 0.00</td>
<td></td>
</tr>
</tbody>
</table>

$P' = 1.150 \text{ kN}$

5.4 RESULTS AND DISCUSSION OF SINUSOIDAL SHAKEABLE TESTS OF WALLS S5 TO S8

5.4.1 General Overall Behaviour

Test Walls S5 to S8 were subjected to a sequence of shakeable tests having sinusoidal excitation of 1.5 Hz, as summarised in Table 4.10. Base shear versus total deflection hysteresis loops were constructed from the recorded response acceleration versus time and response displacement (Fig. 4.18: gauge A1) versus time traces. The base shear $V_{acc}$ was calculated as $MY(t)$ where $M = 4850 \text{ kg}$ is the inertia mass (including concrete blocks, inertia mass support frame and top loading channel beam, Fig. 4.15), and $Y(t)$ is the absolute response acceleration of the inertia mass.

![Diagramatic View of Shakeable Tests](image)

Fig. 5.17 Diagramatic View of Shakeable Tests.
Generally, the base shear $V_{\text{cell}}$, recorded by the load cell mounted between the test wall and the shaketable (Fig. 4.15) was 5% to 10% greater than the base shear $V_{\text{acc}}$ calculated from the response acceleration. Possible reasons for the observed discrepancies are:

1. The calibration of the load cell, which was determined using a slow (quasi-static) loading rate, was affected by dynamic loading.

2. The inertia force of the bottom base beam (Fig. 4.15) contributed towards the base shear $V_{\text{cell}}$ recorded by the load cell. With reference to the diagramatic view of the testing arrangement in Fig. 5.17, the base shear $V_{\text{cell}}$ was the sum of the response inertia force $M\ddot{Y}(t)$ and the inertia force of the base beam $m\ddot{y}_g(t)$, where $m$ is the mass of the base beam and $\ddot{y}_g(t)$ is the shaketable acceleration. For the initial tests on each wall, the frequency ratio $\omega_g/\omega_{\text{nat}}$ is approximately 0.5 (where $\omega_g$ is the shaketable frequency and $\omega_{\text{nat}}$ is the natural frequency of the wall), and the calculated phase angle $\phi$ of the wall response to the shaketable motion is approximately zero (see Fig. 4.20). Consequently, the inertia force of the base beam will act in the same direction as the response inertia force giving $V_{\text{cell}} > V_{\text{acc}}$. As the stiffness of the wall degrades, however, the frequency ratio $\omega_g/\omega_{\text{nat}}$ tends towards unity, and the phase angle $\phi = \pi/2$ (see Fig. 4.20). Consequently, at the maximum response acceleration $\ddot{Y}_{\text{max}}$, the shaketable acceleration, and hence the inertia force $m\ddot{y}_g(t)$ of the base beam, will tend towards zero giving $V_{\text{cell}}$ approximately equal to $V_{\text{acc}}$.

It was not considered necessary to further investigate the 5% to 10% discrepancies between base shear readings, and the base shear $V_{\text{acc}}$ has been used in the presentation of the following results.
Fig. 5.18  Base Shear versus Total Deflection Response Hysteresis Loops for Wall S5.

5.4.1.1  Wall S5

Figure 5.18 presents the base shear versus total deflection hysteresis behaviour of Test Wall S5. Only an envelope curve of the wall response to Tests 1 and 2 is shown in Fig. 5.18. The wall remained essentially elastic during Tests 1 and 2. Stiffness degradation of the wall during Test 3 led to a decrease in the overall wall natural frequency, and this led to the increase in the displacement demand on the wall (Fig. 4.20). The wall deflection record was lost at displacements greater than 60 mm during Test 3, due to an instrument fault. Peak base shear of 41 kN (1.8 V_e) was obtained during Test 3, and this was about 25% greater than the ultimate strength of Walls S1 and S3 (which were identical to Wall S5 except that they were subjected to quasi-static loading). Vertical displacements of the hold-down connections accounted for up to 3 mm of the total wall deflection.
Plate 5.8 WALL S5: at the end of test.

Plate 5.9 WALL S6: failure of sheathing nailing.

Plate 5.10 WALL S7: at the end of test.
$\Delta_n$ during Test 1 and 2, and 6 mm during Test 3. Failure of the wall occurred during Test 3 when the nail heads pulled through the plywood along the east framing stud to plywood connection, as shown in Plate 5.8.

![Graph showing base shear versus deflection response hysteresis loops for Wall S6.](image)

**Fig. 5.19** Base Shear versus Total Deflection Response Hysteresis Loops for Wall S6.

### 5.4.1.2 Wall S6

Wall S6 was identical to Wall S5 except for intentionally loose fitting hold-down connections, which themselves caused approximately 27 mm initial slackness in the force-deflection response of the wall. Summary base shear versus deflection hysteresis loops for Wall S6 are presented in Fig. 5.19. Only an envelope of the wall response to Test 1 and 2 is shown in Fig. 5.19. The wall response to Test 1 and 2 was very stable.
with the wall remaining essentially elastic outside the central "slackness region". Test 3 showed some stiffness degradation and "pinching" of the hysteresis loops. Maximum base shear of 45kN (2.0 V_E) was obtained during Test 3, and this is about 10% greater than the maximum base shear recorded for Wall S5. Wall deflections in excess of 90 mm, and further stiffness degradation occurred during Test 4. Approximately 30 to 40 mm of the total wall deflection Δ_T arose from vertical displacements of the hold-down connections. Failure of Wall S6 was similar to Wall S5 with the nail heads pulling through the plywood along the connection between centre framing stud and plywood, as shown in Plate 5.9.

**Fig. 5.20** Base Shear versus Total Deflection Response Hysteresis Loops for Wall S7.
5.4.1.3 Wall S7

Wall S7 incorporated hold-down connections that were designed to a dependable yield strength of 31 kN (based on 275 MPa yield stress), and in this way designed to limit the maximum base shear imposed on the wall. Summary base shear versus deflection hysteresis loops for Wall S7 are presented in Fig. 5.20. Only an envelope of the wall response to Tests 1 and 2 is shown. The wall hysteresis loops remained stable up to and including Test 2, but increased displacements and stiffness degradation developed during Test 3 and 4. During Test 4, the 12 mm diameter tie of the hold-down connection (Fig. 4.10) yielded in tension, thus limiting the maximum base shear to 36 kN. This was about 15% greater than the dependable strength of 31 kN, and was due to the 12 mm dia. tie having strength greater than its dependable strength.

Nevertheless, the maximum base shear of wall S7 was less than the base shear of Walls S5 and S6, where the ultimate strength was controlled by the strength of the sheathing nails. Figure 5.21 shows the load vs vertical displacement behaviour of the east and west hold-down connections. Yielding of the tie at 36kN resulted in uplift of approximately 5 mm at the base of the wall.

Despite the hold-down connections yielding in tension, the wall failed through the sheathing nails pulling through the plywood along the centre framing stud to plywood connection, as shown in Plate 5.10. Wall lozenge deflection was less than $\Delta_y = 45$ mm at failure, and this represents less than 8 mm sheathing nail slip. This nail slip is significantly less than the nail slip at nail failure during the other test walls and in the nailed joint tests described in Chapter 3. The reason for this, however, is unknown.
Fig. 5.21 Base Shear versus Vertical Anchorage Deflection Hysteresis Loops for Wall S7.
Fig. 5.22 Base Shear versus Total Deflection Response Hysteresis Loops for Wall SB.
Plate 5.11 WALL S8: Condition of framing stud to sheathing connection at the end of test.

(a) North view

(b) South view

Plate 5.12 WALL S8: Perpendicular-to-grain tension failure of bottom framing plate.
5.4.1.4 Wall S8

Wall S8 was fabricated using the framing from Wall S7 and the 12 mm thick plywood from Wall S4. Sheathing nails were hand driven between the previous nail holes in the framing and sheathing. Testing was conducted immediately after fabrication. The base shear versus total deflection hysteresis loops are presented in Fig. 5.22, where only an envelope of the wall response is shown for Tests 1 to 6. Wall behaviour remained essentially elastic up to and including Test 6. Progressive stiffness degradation and pinching of the hysteresis loops occurred during Test 7, with peak base shears increasing to 60 kN (2.6 \( V_E \)). This was approximately 40% greater than the quasi-static ultimate strength of Wall S4. The vertical displacements of the hold-down connections were generally less than 10 mm. Wall S8 failed during Test 8 when the sheathing nails withdrew from the connection between the east framing stud and the plywood as shown in Plate 5.11. Following nail point withdrawal along the framing stud, sheathing nail force components caused local perpendicular-to-grain tension failure of the bottom framing plate, as shown in Plate 5.12. Wall deflections \( \Delta_T \) in excess of 90 mm were sustained before failure.

5.4.1.5 Natural Period of Vibration \( T \)

The natural period of vibration of the wall is plotted in Fig. 5.23 against the maximum response deflection obtained during harmonic excitation. The Fig. 5.23 natural period for each test was established from the response acceleration trace, as the time it took to complete one oscillation immediately after the shaketable excitation had ceased. The natural period of nonlinear oscillators, however, is amplitude dependent and the period shown in Fig. 5.23 should not be taken as the natural period during harmonic excitation. Nevertheless, there is a strong correlation between the free vibration period and the recorded maximum response deflection with periods ranging from less than 0.4 seconds for elastic response, to 0.9 seconds after stiffness degradation had occurred.
Fig. 5.23 Period plotted against Maximum Response Deflection.

Fig. 5.24 Damping Ratio plotted against Maximum Response Deflection.
5.4.1.6 Equivalent Viscous Damping Ratio λ

The equivalent viscous damping ratio λ is plotted in Fig. 5.24 against the maximum response deflection obtained during harmonic excitation. The equivalent viscous damping ratio λ was determined by the logarithmic decrement [95]. The damping ratio λ increased from about 10% during small wall deflections up to about 40% for wall deflections of 80 mm or more. During small wall deflections, the wall remained essentially elastic, and the 10% damping ratio corresponded to elastic damping. At larger wall deflections, the load-deflection response became nonlinear and stiffness degradation occurred. Consequently, the value of 40% damping included the contribution of hysteretic damping arising from energy dissipation within the load-deflection hysteresis loops. Wall S8 exhibited significantly higher damping (λ > 20%) than the other walls during wall deflections less than 20 mm, but exhibited similar damping at greater wall deflections. The reason for the higher initial damping levels of Wall S8 are attributed to the initially higher friction forces at the plywood to framing interface. These friction forces were higher in Wall S8 compared with Walls S5 to S7 because the sheathing nails within Wall S8 (Wall S8 was tested immediately after fabrication) applied larger clamping forces to the joint members than those in the other walls. At a wall deflection between 20 and 30 mm, the nail point was withdrawn from the framing, and the clamping force of the sheathing nails was independent of any initial clamping force. Therefore the damping force was similar for all the walls tested (Fig. 5.24).
Fig. 5.25 Comparison of Parent Base Shear versus Lozenging Deflection curves for Walls S1 and S3 (Subjected to Quasi-Static loading), and Walls S5 and S6 (Subjected to Dynamic loading).

5.4.2 Comparison of Wall Responses

The parent load versus lozenging deflection curves for Walls S1 and S3 (quasi-static) and Walls S5 and S6 (dynamic) are compared in Fig. 5.25. Walls subjected to dynamic loading exhibited enhanced stiffness and strength properties compared to walls subjected to quasi-static loading. The strength of the dynamically excited walls was 34% higher than that of the quasi-statically loaded walls. This difference can be attributed to:

1. Variability of sheathing nail strength
2. Increased sheathing nail strength due to dynamic loading effects, as discussed in Section 3.6, and
3. Damping forces.
Fig. 5.26 Comparison of Envelope of Base Shear versus Lozenging Deflection curves for Walls S5 to S7 during shaketable amplitude Tests $y_g = 15\, \text{mm}, 22.5\, \text{mm}$ and $30\, \text{mm}$. 
The envelopes of the base shear versus lozenging deflection hysteretic response of Walls S5 to S7 are compared in Fig. 5.26 for the 15 mm, 22.5 mm, and 30 mm shaketable amplitude tests. The comparative performances of Walls S5 (no slackness) and S6 (with slackness) usefully illustrates the effects of slackness on the dynamic response of the wall. Wall S6 sustained significantly larger base shears and lozenging deflections than Wall S5 during each amplitude test. The larger base shear demands are attributed to the slackness within the overall response of Wall S6 decreasing the overall natural frequency $\omega_{\text{nat}}$. The decreased natural frequency caused the frequency ratio for Wall S6 to be closer to resonance than that of Wall S5 (Fig. 4.20), and this in turn caused the increase in base shear demands.

Wall S7 was identical to Wall S5 except that the weaker hold-down connections of Wall S7 yielded in tension at a base shear of 36kN. The lozenging response of Wall S5 and S7 was very similar up to and including the 30 mm shaketable amplitude test (see Fig. 5.26). Comparison of Wall lozenging responses after yielding of the hold-down connection of Wall S7 occurred was not possible due to the premature failure of Wall S7 sheathing nailing at a lozenging deflection of $\Delta_k = 45$ mm.

5.4.3 **Comparison with Theory**

5.4.3.1 **Time History Analysis**

In this section, the observed experimental response of Walls S5 to S8 is compared with the predicted response given by a time history computer analysis. Details of the analysis, including the pinching hysteretic model that was developed to idealise the load-deflection hysteretic characteristics of shear walls, are given in the following Chapter (Section 6.2).

The hysteretic model is shown in Fig. 5.27. The initial loading path follows the initial stiffness $k = k_0$ up to a yield load
and then follows a reduced stiffness \( k = r_1 k_0 \) and \( k = r_2 k_0 \). After load reversal, the loading path follows the prescribed pinching stiffness, as shown in Fig. 5.27. For each time step the relative wall displacement \( y(t) \) is given from the expression,

\[
M\dddot{y}(t) + c\ddot{y}(t) + ky(t) = -M\dddot{y}_g(t) \tag{5.1}
\]

where \( M \) is the inertia mass, \( \dddot{y}_g(t) \) is the ground acceleration and \( c \) is the viscous damping coefficient. The wall stiffness \( k \) is updated at each time step in accordance with the load-deflection hysteretic model shown in Fig. 5.27.

---

**Fig. 5.27** Hysteretic Model Idealising the Load-Deflection Hysteretic Loops of Plywood Sheathed Shear Walls.

Of particular interest in matching the theoretical response to the experimental results is the magnitude of the viscous damping coefficient \( c \) to be used in the theoretical model (Eq. 5.1). The viscous damping coefficient \( c \) is expressed in the following
discussion as the damping ratio $\lambda = c/c_{cr}$ where $c_{cr} = 2\omega_{nat}M$ is the critical viscous damping coefficient and $\omega_{nat} = \sqrt{k_0/M}$ is the natural frequency.

Parameters controlling the hysteretic model (Fig. 5.27) were initially chosen to model the quasi-static load-deflection hysteresis loops of the nominally identical Walls S1 and S3. Figure 5.28 tabulates the fitted control parameters, and compares the resulting hysteretic model with the quasi-static response of Wall S3. The accuracy of the idealisation is clearly shown by the comparison.

![Diagram](image)

**Fig. 5.28** Comparison of Proposed Load-Deflection Hysteretic Model with the Load-Deflection Hysteretic Loops of Wall S3.
Fig. 5.29 Comparison of Experimental Total Deflection versus Time, Base Shear versus Time, and Base Shear versus Total Deflection Response Curves of Walls S5 and S6 with those Predicted by the Proposed Time History Model. Hysteretic Model Calibrated from Quasi-Static Load-Deflection Hysteresis Loops.
Wall S6

Fig. 5.29 Continued.
Fig. 5.29 Continued.
Fig. 5.29 Continued.
The input ground accelerations $y_g(t)$ were obtained for each test by digitizing the experimental shake-table acceleration versus time record (see Fig. 4.21).

The theoretical base shear $V_{\text{base}}$ is, from Eq. 5.1, given by,

\[ V_{\text{base}} = \ddot{Y}(t) = c\dot{y}(t) + ky(t) \]  \hspace{1cm} (5.2)

where $\dot{Y}(t) = y(t) + y_g(t)$ is the absolute response deflection. In this way, the theoretical base shear is compared directly with the experimental base shear $V_{\text{acc}}$ of the test walls, (see Section 5.4.1).

At the end of each analytical amplitude test run the degraded condition of the hysteretic model was recorded and stored as the initial conditions for the subsequent amplitude tests.

Figure 5.29 compares the experimental and theoretical deflection vs time, base shear vs time, and base shear vs deflection response curves for each amplitude test of Walls S5 and S6. Results from the theoretical analysis show:

1. During Test 1 the model remained elastic, and the deflection response $y(t)$ and base shear response $V_{\text{base}}$ were insensitive to the damping ratio $\lambda$. This is because the response is insensitive to damping $\lambda$ for frequency ratios $\omega_g / \omega_{\text{nat}}$ less than about 0.5, as shown by the theoretical response curves in Fig. 4.20. The 8% to 10% damping used in the theoretical analysis for Test 1, shown in Fig. 5.29, was determined from the logarithmic decrement values given in Fig. 5.24.

2. During subsequent amplitude tests, the wall stiffness degraded and this decreased the overall natural frequency of the wall, causing the frequency ratio $\omega_g / \omega_{\text{nat}}$ to tend towards unity. Consequently, the response became more sensitive to damping (see Fig. 4.20). A damping ratio $\lambda$ of about 5% was required to match the experimental deflection vs time trace, as shown in Fig. 5.29.
3. By modelling the load-deflection hysteretic response from quasi-static tests and incorporating a damping ratio $\lambda$ of about 5%, a good prediction of the experimental deflections was obtained, but peak base shears were generally under-estimated by 10% to 20% (Fig. 5.29).

The analytical time history model under-estimates the experimental base shear because the viscous damping model does not predict the strength enhancement due to dynamic loading as shown in Fig. 5.25. The viscous damping term $c \dot{y}(t)$ used in the analysis typically produces damping forces that are at maximum at small wall deflections, and nearly zero at the maximum deflection within each cycle.

Table 5.12 Control Parameters for Base Shear versus Deflection Hysteresis Model.

<table>
<thead>
<tr>
<th>WALL</th>
<th>$V_y$ (kN)</th>
<th>$V'_y$ (kN)</th>
<th>$V_{os}$ (kN)</th>
<th>$X_0$ (Kg/m)</th>
<th>$r_1$</th>
<th>$r_2$</th>
<th>$r_{un}$</th>
<th>$\alpha$</th>
<th>$\beta$</th>
<th>$\Delta_{pyap}$ (m)</th>
<th>$\Delta_{bgap}$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S5</td>
<td>20</td>
<td>38</td>
<td>5</td>
<td>2100</td>
<td>0.340</td>
<td>-0.100</td>
<td>1.0</td>
<td>0.4</td>
<td>1.10</td>
<td>0.000</td>
<td>-0.000</td>
</tr>
<tr>
<td>S6</td>
<td>40</td>
<td>43</td>
<td>2.5</td>
<td>2100</td>
<td>0.045</td>
<td>-0.045</td>
<td>1.0</td>
<td>0.4</td>
<td>1.10</td>
<td>0.010</td>
<td>-0.017</td>
</tr>
<tr>
<td>S7</td>
<td>20</td>
<td>35</td>
<td>5</td>
<td>1700</td>
<td>0.390</td>
<td>0.000</td>
<td>1.0</td>
<td>0.4</td>
<td>1.10</td>
<td>0.000</td>
<td>-0.000</td>
</tr>
<tr>
<td>S8</td>
<td>30</td>
<td>60</td>
<td>5</td>
<td>3000</td>
<td>0.286</td>
<td>-0.056</td>
<td>1.0</td>
<td>0.4</td>
<td>1.10</td>
<td>0.000</td>
<td>-0.000</td>
</tr>
</tbody>
</table>

The base shear-deflection response can be predicted more accurately by matching the hysteretic model directly to the experimental base shear-deflection hysteresis loops (see Figs. 5.18 to 5.22). Table 5.12 tabulates the resulting control parameters for the hysteretic model. Figure 5.30 compares the experimental and theoretical deflection vs time and base shear vs time response curves for Walls S5 to S8. For the reasons discussed above, wall response during Test 1 of Walls S5 to S7 and Tests 1 and Test 2 of Wall S8 was insensitive to damping $\lambda$. As the wall response tended towards resonance, damping values that gave a good match were generally less than 2%. There is good agreement between the experimental and theoretical deflection and base shear response curves (see Fig. 5.30).
WALL S5

Fig. 5.30 Comparison of Experimental Total Deflection versus Time and Base Shear versus Time Curves of Walls S5 to S8 with those Predicted by the Proposed Time History Model. Hysteretic Model Calibrated from Dynamic Base Shear-Deflection Hysteresis Loops
Fig. 5.30 Continued,
Fig. 5.30 Continued.
WALL S8

Fig. 5.30 Continued.
WALL 88 (Cont.)

Fig. 5.30 Continued.
5.4.3.2 Closed Form Steady State Solution

In this section, the steady state amplitude of Test Walls S5 and S6 is predicted. There were a sufficient number of cycles of harmonic shaketable motion of the first and second tests of Wall S5 to cause a steady state response, see Fig. 5.30. During both of these two tests, the wall remained essentially elastic. The steady state amplitude $A$ of a single degree of freedom elastic oscillator having inertia mass $M$, and excited by a harmonic ground displacement $A_g \sin (\omega_g t)$ (Fig. 5.31(a)) is [104],

$$A = \frac{AD}{g}$$

Where $A_g$ is ground amplitude

and $D = \left(\frac{\omega_g}{\omega_s}\right)^2 \left[1 - \left(\frac{\omega_g}{\omega_s}\right)^2\right]^2 + \left(2\omega_g\right)^2 \left[1 - \left(\frac{\omega_g}{\omega_s}\right)^2\right]^{-1}$

(5.3)

is the dynamic magnification factor,

and $\omega_s = \sqrt{k_s / m}$ is the natural frequency of the elastic oscillator having spring stiffness $k_s$.

Test 1 of Wall S6 was the only other test that had a sufficient number of cycles of harmonic shaketable motion to cause steady state response, see Fig. 5.30. The base shear versus deflection response of Wall S6: Test 1 shown in Fig. 5.19, indicates that the elastic response of the wall incorporated a central "slackness region" (slackness oscillator, Fig. 5.31(a)). A steady state solution for the slackness oscillator subjected to harmonic ground motion has been developed by Dean and Stewart [105]. The undamped natural frequency $\omega_{nat}$ of the slackness oscillator having inertia mass $M$ is,

$$\omega_{nat} = \frac{\omega_s}{1 + \frac{2}{\pi} \left(\frac{A_s}{A - A_s}\right)}$$

(5.4)

where $A_s$ is slackness amplitude (see Fig. 5.31(a))

$\omega_s = \sqrt{k_s / m}$ is the spring stiffness frequency, i.e. the natural frequency of a linear oscillator having the same spring stiffness $k_s$, but without a slackness gap $A_s$. 
Fig. 5.31 Comparison of Theoretical Steady State Frequency Response Curves for Elastic and Slackness Oscillator with Experimental Response points. (a) The Elastic and Slackness Oscillator. (b) Variation of Dynamic Magnification Factor D with Frequency Ratio $\omega_d/\omega_s$ and Slackness Ratio $A_s/A$ for Damping Ratio $\lambda = 10\%$. 
The steady state amplitude $A$ of the slackness oscillator induced by a harmonic ground amplitude $A_g \sin(\omega_g t)$ may be conveniently approximated by that of an equivalent elastic oscillator having the same natural frequency $\omega_{nat}$. The frequency ratio $\omega_g/\omega_{nat}$ in terms of the dynamic magnification factor $D = A/A_g$ for the slackness oscillator is

$$(\omega_g/\omega_{nat})^2 = \frac{1}{(1 - 2\lambda_s^2) \pm \sqrt{(1 - 2\lambda_s^2) - (1 - 1/D^2)}}$$  \hspace{1cm} (5.5)$$

where $\lambda_s = \lambda_{nat}$ is the equivalent viscous damping ratio of the slackness oscillator, which results in a damping force $c\dot{y}(t) = 2\lambda_{nat}My(t)$ for the slackness oscillator identical to that for the linear elastic (non slack) oscillator.

The dynamic magnification factor $D = A/A_g$ is plotted in Fig. 5.31(b) against frequency ratio $\omega_g/\omega_s$ for damping ratio $\lambda = 10\%$ and for two slackness ratios $A_s/A_g = 0$ (Eq. 5.3) and $A_s/A_g = 1.8$ (Eq. 5.4 and 5.5). The Fig. 5.31(b) response curves correspond to the experimental slackness ratios $A_s/A_g$ for Wall S5, and $A_s/A_g = 1.8$ for Wall S6.

Also plotted in Fig. 5.31(b) are the experimental points of the three tests in which there were a sufficient number of shake table cycles to induce steady state response. The natural spring frequency $\omega_s$ of Wall S5: Test 1 and 2 was determined from the free vibration displacement versus time trace after each test. The natural spring frequency $\omega_s$ of Wall S6: Test 1 was assumed to be the same as Wall S5: Test 1. There is very good agreement between the experimental points and the theoretical response curves.
5.5 RESULTS AND DISCUSSION OF EL CENTRO 1940 NS SHAKE TABLE 
TESTS OF WALLS S9 TO S11

Test Walls S9 to S11 were subjected to the El Centro 1940 NS component ground motion record. In order to observe the effects of stiffness degradation on the wall response, each wall was subjected to the ground motion record 3 times or until failure. The base shear coefficient $C_b$ is introduced as $C_b = V_E / Mg$ where $V_E$ is the design earthquake load (Table 4.1), $M = 4850$ kg is the inertia mass, and $g = 9.81$ (m/sec$^2$) is the acceleration of gravity. Table 5.13 tabulates the design earthquake load $V_E$ and the base shear coefficient $C_b$ for each wall.

Table 5.13 Base Shear Coefficient.

<table>
<thead>
<tr>
<th>Wall</th>
<th>Design Earthquake load $V_E$ (kN)</th>
<th>Base Shear Coefficient $C_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S9</td>
<td>23.0</td>
<td>0.48</td>
</tr>
<tr>
<td>S10</td>
<td>23.0</td>
<td>0.48</td>
</tr>
<tr>
<td>S11</td>
<td>11.5</td>
<td>0.24</td>
</tr>
</tbody>
</table>

5.5.1 Observed Behaviour

5.5.1.1 Wall S9

Wall S9 was very similar to Wall S6, also having loose fitting hold-down connections permitting an initial slackness of 30 mm in the lateral load-deflection response of the wall. Figure 5.32 shows the first 6 seconds of the recorded base shear versus total deflection hysteretic response of Wall S9 during Test 1 and 2. The El Centro ground displacement record was increased by 20% during Test 2.
Fig. 5.32 Base Shear versus Total Deflection Response Hysteresis Loops for Wall S9.

The wall remained essentially elastic outside of the slackness region during Test 1. On the first major positive excursion during Test 2, the east hold-down connection failed. Inspection of the connection at the end of the test showed that a nut had been inadvertently omitted from one of the two ties, and the thread on the other tie had stripped during the test. No further
tests of Wall S9 were undertaken, even though the wall itself was not damaged.

5.5.1.2 Wall S10

Figure 5.33 shows the first 6 seconds of the base shear vs total deflection hysteretic response of Wall S10 during Tests 1 to 3. The El Centro displacement record was increased by 20% during Tests 2 and 3. Wall S10 remained essentially elastic during Test 1 and the maximum base shear approximated the design earthquake load $V_E = 23$ kN. Some stiffness degradation of the hysteresis loops occurred during Test 2 and 3. However, the overall wall deflection only increased about 10 mm during each test, and maximum wall deflection did not exceed 40 mm. At the end of the test, the wall was still capable of carrying lateral load.

5.5.1.3 Wall S11

Wall S11 was fabricated using the same framing and sheathing as Wall S10. The old sheathing nails (Wall S10) were removed and new nails were hand driven between the existing nail holes. The wall was tested immediately after fabrication. The first 6 seconds of the response of Wall 11 during Tests 1 to 3 is presented in Fig. 5.34. Shaketable displacements were increased to 1.2 times El Centro magnitudes during Test 3. Stiffness degradation of the hysteresis loops occurred during Test 1. Maximum wall deflection was 20 mm, 40 mm and 60 mm for Tests 1, 2 and 3 respectively (Fig. 5.34). The wall was still capable of carrying lateral load at the end of Test 3.
Fig. 5.33 Base Shear versus Total Deflection Response Hysteresis Loops for Wall S10.
Fig. 5.34  Base Shear versus Total Deflection Response Hysteresis Loops for Wall S11.
5.5.2 Comparison of Walls S9 to S11

The envelope of the base shear versus lozenging deflection response of Wall S9:Test 1 and Wall S10:Test 1 are compared in Fig. 5.35. Figure 5.35 shows very little difference in base shear demand between the two walls despite Wall S9 incorporating 30 mm initial slackness. The similar base shear demand can be explained with reference to the El Centro 1940 NS response spectra shown in Fig. 5.36. The response acceleration of the elastic oscillator of Wall S10, having a fundamental period of vibration of approximately 0.37 seconds, and 10% damping is 0.65g. The equivalent natural period of vibration of the slackness oscillator of Wall S9 is with reference to Eq. 5.4 (noting that $T_o = 2\pi/\omega_{nat}$), $T_o = 1.6T_s = 0.6$ seconds where $T_s = 0.37$ seconds is the natural period of vibration of a linear elastic oscillator having the same spring stiffness $k_s$ as the slackness oscillator, $A_s = 30$ mm and $A = 60$ mm. Figure 5.36 shows that the response acceleration of an elastic responding oscillator having period of 0.6 seconds and 10% damping is very similar to that obtained for Wall S10 having period of 0.37 seconds.

Fig. 5.35 Comparison of the Envelope of Base Shear versus Lozenging Deflection Hysteresis Loops of Walls S9 and S10.
Fig. 5.36 Elastic Response Acceleration as a function of damping and period of vibration for a single degree of freedom elastic oscillator responding to the El Centro 1940 NS ground motion record.

5.5.3 Comparison with Time History Analysis

In this section, the recorded experimental response of Wall S9 and S10 is compared with the predicted response given by time history analysis. The hysteretic model was calibrated from the quasi-static hysteresis loops of Wall S1 and S3, as shown in Fig. 5.28.
The experimental and theoretical displacement vs time response curves for Test 1 of Walls S9 and S10 are compared in Fig. 5.37. Reasonably good agreement between the experimental and theoretical response curves was obtained for a damping ratio of 10%.

**Fig. 5.37** Comparison of Experimental Total Deflection versus Time Response curves of Walls S9 and S10 with that predicted by the Proposed Time History Model using $\lambda = 10\%$. 
5.6 Seismic Design Methodology

5.6.1 General

This section presents a seismic design methodology for timber sheathed shear walls utilising the capacity design procedure. This procedure requires ductile energy-dissipating components to be provided within the structure, and other components in the load path to be provided with reserve strength to prevent the formation of any undesirable failure mechanism. The load resisting components of a shear wall are shown in Fig. 5.38 as a series of links making up the lateral resistance of the wall. The most desirable ductile energy dissipating component or ductile link is the sheathing nailing. The experimental study of nailed sheathing joints, described in Chapter 3, showed that the sheathing nails comprised an ideal ductile component, and that well detailed joints permitted considerable postelastic displacement capacity even when subjected to reverse cyclic loads. It is generally economic to make the other components in the wall, (i.e. the framing members, sheathing and anchorage connections) stronger than the ultimate strength of the sheathing nails so that brittle failure of these components in bending, shear or tension does not occur. The sheathing nailing is a more suitable ductile component than the anchorage connections. Many commonly used anchorage connection details perform poorly when subjected to reverse cyclic loads, exhibiting only limited post elastic deformation capacity and unfavourable failure modes, as described in Chapter 4.

Fig. 5.38 Chain of Resistance of Plywood Sheathed Shear Walls.
In order to achieve a desirable strength hierarchy, it is necessary to know the strength and variability of each component within the wall. Figure 5.39 portrays graphically the strength distribution of the sheathing nailing $P$, and the strength distribution of the non-ductile components $F$. The object of the capacity design procedure is to ensure that at all times $P > F$. This assurance is only possible in terms of the probability or risk that $P > F$, and this is represented by the overlap, region of Fig. 5.39. Determining the magnitude of this overlap, and hence the probability that $P > F$, depends upon the relative positions ($\mu_p$, $\mu_F$) and shape of the probability distributions, however this is beyond the scope of this study.

In the following design method, the strength hierarchy is achieved by ensuring that the dependable strength of the non-ductile components $F_d$ is equal to or larger than the overstrength $P_o$ of the ductile sheathing nailing. The dependable strength is defined here as the 5%ile characteristic strength of the component. In this study the overstrength $P_o$ of the sheathing nailing is taken for convenience as the 95%ile characteristic strength of the nailed sheathing joints described.
in Chapter 3. This leads to an upper bound estimate of the wall overstrength $V_0$, and provides conservative protection against the risk of failure of the non-ductile components. However, because a shear wall contains a large number of sheathing nails, the required protection may be obtained by using a lower characteristic strength value, which thus leads to a more economic shear wall design. The magnitude of this overstrength can only be determined after further experimental studies have been undertaken to assess the design procedure and to ensure that a reduced overstrength value will provide the required protection against brittle shear wall failure.

5.6.2 Shear Wall Strength Equations

In this section, a simplified strength theory for shear walls incorporating sheathing aspect ratio $a = 2$ is described. In the strength theory formulated in Chapter 2, wall strength, framing stud bending moment, joint forces etc. were shown to depend on the inelastic stiffness parameter $K_I$ and the nail slip ratio $r_s$. Although an equation for determining the nail slip ratio $r_s$ was derived in Chapter 2, it had no closed form solution. It is convenient, therefore, to simplify the Chapter 2 strength theory by using an approximation for the nail slip ratio $r_s$. In this section, the nail slip ratio $r_s$ is approximated to produce upper bound values for the framing bending moment, axial load, and joint forces.

5.6.2.1 Nail Slip Ratio for Maximum Framing Stud Actions $r_{s,s}$

In this section, an approximate nail slip ratio $r_s$ is determined to provide an upper bound value for the framing stud bending moment, axial force and horizontal framing joint force. These actions are maximum when the nail slip ratio $r_s$ of the panel is small (Fig. 2.20 and 2.21). The nail slip ratio itself depends on the inelastic stiffness parameter $K_I$, and framing joint separation (Fig. 2.24).
Table 5.14 Typical Range of Inelastic Stiffness Parameter $K_I$.

<table>
<thead>
<tr>
<th>Nominal Size (mm)</th>
<th>Standard Gauged Dimensions (mm)</th>
<th>$I_{x10^6}$ (mm$^4$)</th>
<th>$I_{y10^6}$ (mm$^4$)</th>
<th>$A_{x10^3}$ (mm$^2$)</th>
<th>$A_{y10^3}$ (mm$^2$)</th>
<th>Inelastic stiffness parameter $K_I$</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 x 50</td>
<td>94 x 47</td>
<td>34.61</td>
<td>4.42</td>
<td>0.0030</td>
<td>0.0060</td>
<td>0.0090 0.0119 0.0179</td>
</tr>
<tr>
<td>150 x 50</td>
<td>144 x 47</td>
<td>53.02</td>
<td>6.77</td>
<td>0.0046</td>
<td>0.0091</td>
<td>0.0137 0.0183 0.0274</td>
</tr>
<tr>
<td>100 x 100</td>
<td>94 x 94</td>
<td>138.43</td>
<td>8.84</td>
<td>0.0239</td>
<td>0.0466</td>
<td>0.0716 0.0955 0.1433</td>
</tr>
</tbody>
</table>

$k_s = F_S/8.0\, \text{mm} = 160\, \text{N/mm}$

Table 5.14 tabulates the typical range of the inelastic stiffness parameter $K_I$ for common framing member sizes and nail spacings. For the Table 5.14 range of parameters, a minimum value for the inelastic stiffness parameter can be assumed to be $K_I = 0.001$.

Results from tests reported in Section 5.3 showed that framing joint slip ratios $r_{c,x} = r_{c,y} = 0.2$ were inevitable despite the strong nail plate brackets used to connect the framing members.

For a minimum inelastic stiffness parameter $K_I = 0.001$ and assuming $r_{c,a} = r_{c,y} = 0.2$, a minimum nail slip ratio $r_s$ can be approximated as $r_s = 0.8$ (see Fig. 2.24), the corresponding nail slip ratio for the framing stud $r_{s,s}$ that will induce maximum bending moment and horizontal framing joint force is (from Eq. 2.79).

$$r_{s,s} = \frac{r_s}{1 - r_{c,x}} = \frac{0.8}{1 - 0.2} = 1.0$$

The corresponding nail slip ratio for the framing plate $r_{s,p}$ that will induce maximum axial force in the edge framing stud is (from Eq. 2.78)

$$r_{s,p} = r_s - r_{c,y} = 0.8 - 0.2 = 0.6$$
5.6.2.2 Nail Slip Ratio for Maximum Framing Plate Actions $r_{s,p}$

In this section, the value of the nail slip ratio $r_s$ is determined to produce an upper bound value for the framing plate bending moment and the vertical framing joint force. Framing plate actions are maximum when the nail slip ratio $r_s$ of the panel is a maximum (Fig. 2.19 and 2.21). For a maximum inelastic stiffness parameter $k_i = \infty$ and assuming $r_{c,x} = r_{c,y} = 0.2$, a maximum nail slip ratio $r_s$ is $r_s = 1.6$ (Fig. 2.24). The corresponding nail slip ratio for the framing plate $r_{s,p}$ that will induce maximum bending moment and vertical framing joint force is (from Eq. 2.78)

$$r_{s,p} = r_s - r_{c,y} = 1.6 - 0.2 = 1.4$$

5.6.2.3 Overstrength Equations

Nail strength is shown in Table 2.9 to decrease marginally with increasing framing stud flexibility and framing joint separation. An upper bound value of the wall overstrength may be conveniently estimated by (see Table 2.9)

$$V_o = 1.15 P_o n B / c$$ (5.6)

where $P_o$ is the sheathing nail overstrength.
Table 5.15 Overstrength Equations for Shear Wall incorporating sheathing panel aspect ratio \( \alpha = 2 \).

<table>
<thead>
<tr>
<th>Notation</th>
<th>Overstrength Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Overstrength</td>
<td>( V_o ) ( \frac{1.15 P_o n B}{c} )</td>
</tr>
<tr>
<td>Edge Framing Stud</td>
<td></td>
</tr>
<tr>
<td>Nail slip ratio</td>
<td>( r_{s,s} ) ( 1.0 )</td>
</tr>
<tr>
<td>Bending Moment</td>
<td>( M_{o,s,E} ) ( 24.0 \times 10^{-3} \phi_{om,E} V_o )</td>
</tr>
<tr>
<td>Axial force</td>
<td>( P_{o,s,E} ) ( \frac{1.92 V_o}{n} )</td>
</tr>
<tr>
<td>Framing joint force</td>
<td>( X_{o,E} ) ( \frac{0.23 \phi_{ox,E} V_o}{n} )</td>
</tr>
<tr>
<td>Interior Framing Stud</td>
<td></td>
</tr>
<tr>
<td>Nail slip ratio</td>
<td>( r_{s,s} ) ( 1.0 )</td>
</tr>
<tr>
<td>Bending Moment</td>
<td>( M_{o,s,I} ) ( 48.0 \times 10^{-3} \phi_{om,I} V_o )</td>
</tr>
<tr>
<td>Axial force</td>
<td>( P_{o,s,I} ) ( 0 )</td>
</tr>
<tr>
<td>Framing joint force</td>
<td>( X_{o,I} ) ( \frac{0.46 \phi_{ox,I} V_o}{n} )</td>
</tr>
<tr>
<td>Framing Plates</td>
<td></td>
</tr>
<tr>
<td>Nail slip ratio</td>
<td>( r_{s,p} ) ( 1.4 )</td>
</tr>
<tr>
<td>Bending Moment</td>
<td>( M_{o,p} ) ( 0.015 B V_o )</td>
</tr>
<tr>
<td>Edge Framing joint force</td>
<td>( Y_{o,E} ) ( \frac{0.16 V_o}{n} )</td>
</tr>
</tbody>
</table>

**NOMENCLATURE**

\( \phi_{om} \) = Bending moment overstrength reduction factor, and is plotted in Fig. 5.40 against the inelastic stiffness parameter \( K_r \). \( \phi_{om} \) and \( K_r \) may be further subscripted \( E \) for edge framing stud or \( I \) for interior framing stud.

\( \phi_{ox} \) = Framing joint overstrength reduction factor, and is plotted in Fig. 5.40 against the inelastic stiffness parameter \( K_r \). \( \phi_{ox} \) and \( K_r \) may be further subscripted \( E \) for edge framing stud or \( I \) for interior framing stud.
Overstrength equations for the framing axial force, bending moment and joint forces, based on the assumed nail slip ratios determined in the previous sections are summarised in Table 5.15. The bending moment overstrength reduction factor \( \phi_{o,m} = M_{o,s}/M_{o,s(rigid)} \) and the framing joint overstrength reduction factor \( \phi_{o,x} = X_{o}/X_{o(rigid)} \) are introduced in the Table 5.15 equations, and are plotted in Fig. 5.40 against the inelastic stiffness parameter \( K_f \).

where:

- \( M_{o,s} \) = maximum bending moment in framing stud at wall overstrength \( V_o \)
- \( M_{o,s(rigid)} \) = maximum bending moment in rigid framing studs at wall overstrength \( V_o \)
- \( X_{o} \) = Horizontal framing joint force at wall overstrength \( V_o \)
- \( X_{o(rigid)} \) = Horizontal framing joint force for rigid framing studs at wall overstrength \( V_o \)
Fig. 5.40 The Framing joint overstrength reduction factor $\phi_{o,x}$ and bending moment overstrength reduction factor $\phi_{o,m}$ plotted against the inelastic stiffness parameter $K_I$.

### 5.6.3 Summary of Design Methodology

In this section, a summary of a capacity design procedure for earthquake resistant shear walls is presented.

**Step 1: Sheathing Nail Spacing $c$**

For the specified design earthquake load $V_E$, determine the necessary sheathing nail spacing $c$ assuming elastic shear wall behaviour (Eq. 2.39).

$$c < \frac{nBP_E}{V_E}$$  \hspace{1cm} (5.7)

where $P_E$ is the design earthquake nail load. The actual nail spacing $c$ should satisfy the requirements of Eq. 5.1, since nail
spacing which is smaller than that required will increase the wall overstrength $V_o'$, and therefore the necessary size of the framing, sheathing, and anchorages.

**Step 2:** Sheathing Nail Overstrength $P_o$

Assess the overstrength $P_o$ of the sheathing nails (Eqs. 3.25).

$$ P_o = P \phi_o $$

(5.8)

where $\phi_o$ is the overstrength factor

$$ = 1.2 K_w K_t \left(1.461 + 0.054 \frac{l_n}{d_n} - 0.724 d_n\right) $$

and $l_n$ = nail length (mm)
and $d_n$ = nail diameter (mm)

and the moisture content factor

$$ K_w = 1.0 \text{ green/dry 18\% moisture content} $$
$$ = 1.2 \text{ green/dry 10\% moisture content} $$
$$ = 1.4 \text{ dry/dry 10\% moisture content} $$

and the nail type factor

$$ K_t = 1.0 \text{ plain bright or zinc steel nails} $$
$$ = 1.4 \text{ plain galvanised steel nails} $$

In order to reduce the wall overstrength $V_o'$, and hence the required size of the framing, sheathing and anchorages, the overstrength $\phi_o$ should be made as small as possible. Therefore, for earthquake resistant design, short zinc coated nails are preferable to longer galvanised nails, and green framing is preferable to dry framing.

**Step 3:** Wall Overstrength $V_o$

Derive the wall overstrength $V_o$ (Table 5.15)

$$ V_o = 1.15 P_o n B/c $$

(5.9)

where $c$ is the provided nail spacing.
Step 4: Sheathing Thickness $t_s$

Determine the required sheathing thickness $t_s$ in order to suppress brittle shear failure of the sheathing (from Table 2.10)

$$t_s > V / F_{o,s,n} \cdot B$$

(5.10)

where $F_{o,s,n} = 2.2 \cdot F'_{s}$ is the dependable shear strength of the sheathing as given by NZS 3603: Clause 2.12 [11] and $F'_{s}$ is the NZS 3615:1981 [73] basic permissible shear stress of the sheathing material.

Step 5: Framing Member Sizes

Proportion the edge framing stud members such that at the section of maximum bending moment

$$\frac{f_{o,b}}{F_{d,b}} + \frac{f_{o,t}}{F_{d,t}} < 1.0$$

(5.11)

or at the section of maximum axial tension (bending moment is zero at end of framing stud, see Fig. 2.18)

$$\frac{f_{o,t}}{F_{d,t}} < 1.0$$

(5.12)

where $f_{o,b} = 24.1 \times 10^{-3} \cdot h_{V} / v_{n}$ is the maximum flexural stress (Table 5.15)

$f_{o,t} = 1.92V_{o} / nA$ is the maximum axial stress (Table 5.15)

$F_{d,b} = 2.2 \cdot F'_{b}$ is the dependable flexural strength of framing

$F'_{b} = $ the basic permissible flexural stress of framing

$F_{d,t} = 0.6 \cdot F_{d,b}$ + is the dependable axial tension strength of framing.

$z$ = effective section modulus of framing at the critical section, i.e. subtracting bolt holes etc.

+ Proposed amendment to NZS 3603: Table 2 [11] reduces the basic permissible tensile stress from $F'_{t} = 0.8 \cdot F'_{b}$ to $F'_{t} = 0.6 \cdot F'_{b}$ [106].
A = effective area of framing at the critical section.

ϕ_{o,m} = bending moment overstrength reduction factor (see Fig. 5.40).

ϕ_t = f_{o,t}/f_{o,t(M,max)} = 1 - y_{m,max}/H assuming triangular axial force distribution along stud.

f_{o,t(M,max)} = axial stress in framing stud at the position of maximum bending moment.

y_{m,max} = position of maximum bending moment from end of framing stud (see Fig. 2.22).

If the edge framing stud members are subjected to gravity loads, combined bending and axial compression loading may be more critical than combined bending and tension.

The framing sizes required for the edge framing stud can also be used for the interior framing studs and framing plates, as the latter are not as highly stressed as the edge framing stud. Alternatively, smaller framing sizes could be provided for the interior framing studs and framing plates, as required by the equations tabulated in Table 5.15.

Step 6: Framing Joint Connections

Determine the framing joint forces \( X_{o,E} \) and \( Y_{o,E} \) at the corner edge between the edge framing stud and the top framing plate.

\[
X_{o,E} = \frac{0.23 \, \phi_{o,x} \, V_o}{n} \quad (5.13)
\]

\[
Y_{o,E} = \frac{0.16 \, V_o}{n} \quad (5.14)
\]
where $\phi_{o,x}E$ is the framing joint overstrength reduction factor of the edge framing stud for the inelastic stiffness parameter $K_{I,E}$ (see Fig. 5.40). Assuming the linear interaction diagram advanced by De Bonis and Bodig (Mhlebeck [29]), select the framing joint connection such that

$$\frac{X_{o,E}}{F_{d,x}} + \frac{Y_{o,E}}{F_{d,y}} \leq 1.0 \quad (5.15)$$

where $F_{d,x}$, $F_{d,y}$ are the dependable strengths of the selected connection in the direction of framing joint force $X_{o,E}$ and $Y_{o,E}$ respectively.

Calculate the horizontal framing joint force $X_{o,I}$ for the interior framing plate connection,

$$X_{o,I} = 0.46 \phi_{o,x} V_o/n$$

where $\phi_{o,x,I}$ is the framing joint overstrength reduction factor of the interior framing stud for inelastic stiffness parameter $K_{I,I}$ (Fig. 5.40).

Choose the interior framing joint connection such that

$$\frac{X_{o,I}}{F_{d,x}} \leq 1.0$$

**Step 7: Anchorage Connections**

Design the shear anchorage connection between the bottom framing plate and foundations to resist overstrength $V_o$. Design the vertical anchorage connections located at both edge framing studs to resist overstrength forces $2V_o/n$ (Fig. 5.41). When the framing stud butts into the inside face of the framing plate (Fig. 5.41), check that the perpendicular-to-grain bearing stress under the compression edge framing stud at the design earthquake load $V_E$ does not exceed the dependable perpendicular-to-grain bearing stress $F_{d,p}$. 
\[ P_{d,p} < 1.92 \frac{V_e}{n_A} \]

where \( P_{d,p} = 2.2F_p \) and \( F_p \) is the basic permissible perpendicular-to-grain bearing stress.

---

**Fig. 5.41** Construction Details of 3.6 m long plywood sheathed shear wall example.

5.6.4 **Illustrative Example**

In this section, the design procedure described in section 5.6.3 is used to design the 3.6 m long by 2.4 m high shear wall shown in Fig. 5.41. The wall is required to resist a design earthquake load of 30 kN, and does not carry gravity loads. The design begins by selecting 12 mm thick plywood sheathing which is fastened to one side of 100 x 50 green gauged framing members (94 x 47 Nett) with 40 x 2.89 mm diameter zinc coated nails. All framing is No. 1 framing grade Pinus Radiata. Plywood is D-D grade Pinus Radiata. The NZS3603:1981 [11] permissible wind/seismic nail load for a 2.89 mm dia. nail is \( P_{ws} = 560 \text{ N}. \)
\[ P_E = 1.25 \text{ P}_{ws} = 1.25 \times 560 = 700 \text{ N} \]

Nail Spacing \( c \)

The required nail spacing \( c \), from Eq. 5.7, is

\[ \frac{nBP_E}{V_E} < \frac{3 \times 1200 \times 700}{30 \times 10^3} = 84 \text{ mm} \]

say \( c = 75 \text{ mm} \)

Nail Overstrength \( P_o \)

From Eq. 5.3 the overstrength factor \( \phi_o \) is

\[ \phi_o = 1.2 K_o K_t (1.461 + 0.054 L_n - 0.724 d_n) \]

Timber condition is green/dry with expected in-service moisture content of 18% giving \( K_o = 1.0 \). Nails are zinc coated giving \( K_t = 1.0 \).

\[ \phi_o = 1.2 (1.461 + 0.054 \times 40 - 0.724 \times 2.89) = 1.83 \]

\[ P_o = \phi_o P_E = 1.83 \times 700 = 1280 \text{ N} \]

Wall Overstrength \( V_o \)

Based on the provided nail spacing \( c = 75 \text{ mm} \), the wall overstrength is given from Eq. 5.9,

\[ V_o = \frac{1.15 P_o nB}{c} = \frac{1.15 \times 1280 \times 3 \times 1200}{75} = 70.7 \text{ kN} \]

Plywood Thickness \( t_s \)

Minimum sheathing thickness to suppress brittle shear failure is from (Eq. 5.10)
Although the plywood thickness could be reduced to 7.5mm, the thicker 12 mm thick plywood is recommended because of the significantly improved seismic behaviour of 2.89 mm dia. nails driven through the thicker plywood.

Framing Members

Edge Framing Stud:

Check tension for 100 x 50 stud (Eq. 5.12).

\[ f_{o,t} = \frac{1.92 V_o}{nA} = \frac{1.92 \times 70.7 \times 10^3}{3 \times 4.42 \times 10^3} = 10.2 \text{ MPa} \]

\[ F_{d,t} = 0.6 F'_{b,t} = 0.6 \times 2.2 \times F'_{b} = 0.6 \times 2.2 \times 6.0 = 8.0 \text{ MPa} \]

\[ f_{o,t} > F_{d,t}, \text{ clearly the 100 x 50 section size is insufficient.} \]

Therefore try 150 x 50 framing member; section modulus for 150 x 50 section is \( z = 53.02 \times 10^3 \text{ mm}^3 \) (see Table 5.14).

Calculate tension and bending stress for 150 x 50 stud.

From Table 5.13 we have,

\[ f_{o,b} = \frac{24 \times 10^{-3} \phi_{o,m} HV}{nz} = \frac{24 \times 10^{-3} \times 0.245 \times 2400 \times 70.7 \times 10^3}{3 \times 53.02 \times 10^3} = 6.3 \text{ MPa} \]

\[ f_{o,t} = \frac{1.92 V_o}{nA} = \frac{1.92 \times 70.7 \times 10^3}{3 \times 6.77 \times 10^3} = 6.7 \text{ MPa} \]

\[ F_{d,b} = 2.2 F'_{b} = 2.2 \times 6.0 = 13.2 \text{ MPa} \]

\[ F_{d,t} = 0.6 F_{d,b} = 8.0 \text{ MPa} \]

Check tension.
\[ \frac{f_{o,t}}{F_{d,t}} = \frac{6.7}{8.0} < 1.0 \quad \text{therefore OK.} \]

The position of maximum bending moment is, from Fig. 2.22, for \( K_I = 0.01, Y_{M,max} = 300 \text{ mm from end of framing stud.} \)
Assuming a triangular distribution of axial force along the stud, then

\[ \psi_t = 1 - \frac{Y_{M,max}}{H} = 1 - \frac{300}{2400} = 0.875 \]

Check combined bending and tension at position of maximum bending moment.

\[ \frac{f_{o,b}}{F_{d,b}} + \frac{\psi_t f_{o,t}}{F_{d,t}} = \frac{6.3}{13.2} + \frac{0.87 \times 6.7}{8.0} = 1.2 > 1.0 \]

showing that the 150 x 50 section is overstressed.
Therefore try 100 x 100 framing member.

Framing stud maximum bending stress \( f_{o,b} \) and axial stress \( f_{o,t} \) is plotted against nail spacing \( c \) and framing member size in Fig. 5.42. For 100 x 100 framing member, we have

\[ f_{o,t} = 5.1 \text{ MPa} \]
\[ f_{o,b} = 5.4 \text{ MPa} \]

Check combined bending and tension at position of maximum bending moment (Eq. 5.11).

\[ \frac{f_{o,b}}{F_{d,b}} + \frac{\psi_t f_{o,t}}{F_{d,t}} = 0.97 < 1.0 \]

Therefore, section size OK.
Fig. 5.42 Actions on the edge framing stud members plotted against nail spacing $c$ and framing member size. (a) Maximum axial stress $f_{ot}$. (b) Maximum flexural stress $f_{b,E}$. 
Fig. 5.43 Framing Joint Forces for the Edge Framing stud members plotted against nail spacing $c$ and framing member size.  
(a) Horizontal Framing Joint Force.  
(b) Vertical Framing Joint Force.
Framing Joint Connections

Exterior joint:

Horizontal framing joint force (Eq. 5.13)

\[
X_{o,E} = \frac{0.23 \phi_{ox} V_0}{n} = \frac{0.23 \times 0.75 \times 70.7}{3} = 4.1 \text{ kN}
\]

Vertical framing joint force (Eq. 5.14)

\[
Y_{o,E} = \frac{0.16 V_0}{n} = \frac{0.16 \times 70.7}{3} = 3.8 \text{ kN}
\]

The framing joint force is plotted against sheathing nail spacing and framing member size in Fig. 5.43. It is common practice to butt the framing stud into the inside face of the framing plate, and connect the two members with nails. The nails resist horizontal force \(X_{o,E}\) through shear, and vertical force \(Y_{o,E}\) through nail shank withdrawal. Taking the dependable shear strength of the nailed joint as \(P_d = 4.15 P_B^r\) and in withdrawal as \(F_{d,w} = 2.5 F_w\) where \(P_B^r\) and \(F_w\) are the NZS3603:1981 [11] basic permissible nail loads, then for two 150 x 6.0 nails driven into end grain (end grain factor for nails in shear = 0.67)

\[
P_d = 4.15 \times 0.67 \times 2.0 \times 0.721 = 4.0 \text{ kN}
\]

\[
F_{d,w} = 2.5 \times 2.0 \times 8.58 \times (150 - 94) \times 10^{-3} = 2.4 \text{ kN}
\]

From Eq. 5.10

\[
\frac{X_{o,E}}{P_d} + \frac{Y_{o,E}}{F_{d,w}} = \frac{4.1}{4.0} + \frac{3.8}{2.4} = 2.6 > 1.0
\]

therefore use nail plate brackets at corners. Try two multigrip connectors (see Table 5.3),

thus
\[
F_{d,x} = 12.2 \text{ kN}
\]

\[
F_{d,y} = 13.0 \text{ kN}
\]
and \[
\frac{X_{o,E}}{F_{d,x}} + \frac{Y_{o,E}}{F_{d,y}} = \frac{4.1}{12.2} + \frac{3.8}{13.0} = 0.63 < 1.0 \quad 0.63 < 1.0
\]

therefore multigrip connectors [97] are satisfactory.

**Interior joint:**

For two nail lines along interior framing stud \( c = 37.5 \text{ mm} \),

hence (Fig. 5.43) \( X_{o,I} = 6.9 \text{ kN} < F_{d,x} = 12.2 \text{ kN} \)

therefore use two multigrip connectors at interior framing joints.

---

**Fig. 5.44** Vertical Anchorage Connection. (a) Bolted hold-down connection. (b) Perpendicular-to-grain bearing stresses in framing plate under edge framing stud. (c) Dummy stud connected to edge framing stud. (d) Edge framing stud extended to the full height of the wall.
Fig. 5.45 Bending moment Diagram for Edge Framing Stud Member with eccentric hold-down connection.

Anchorage Connections

The anchorage connections must be strong enough to develop the strength of the sheathing nailing if brittle failure of the anchorage connections is to be prevented. Shear wall failures during the San Fernando earthquake [7] were almost entirely due to local failure of the anchorage connections. While bolts or nail plate connectors may be easily provided to resist the horizontal shear $V_o$, the hold-down anchorage connections transferring uplift $2V_o/n$ from each edge framing stud to the wall supports require special attention.

A commonly used vertical anchorage connection is shown in Fig. 5.44 (a). The eccentric tie force of the rigid steel bracket with the framing stud (Fig. 5.44(a)) will subject the already
highly stressed edge framing stud to additional bending moment, as illustrated in Fig. 5.45. It is generally more economic, therefore, to position two similar hold-down connections symmetrically about the framing stud. Nailed connections are preferred to bolted connections because the bolt holes reduce the tensile and bending strength of the edge framing stud at the position of the anchorage connection.

When the framing stud butts into the inside face of the framing plate, the compression framing stud induces perpendicular-to-grain bearing stresses in the framing plate, as shown in Fig. 5.44(b). If the perpendicular-to-grain bearing strength of the framing plate is less than the vertical anchorage force $2V_E/n$, wall strength will be controlled by bearing strength of the framing plate and will be less than the required design earthquake strength $V_E$. In such cases, a dummy stud adjacent to the edge framing stud may be added to reduce the bearing stresses or the framing stud may be extended to the full height of the wall, as detailed in Fig. 5.44 (c) and (d).

5.7 SUMMARY OF EXPERIMENTAL RESPONSE

1. General Overall Behaviour

The walls remained essentially elastic up to total wall deflections of 15 mm or so without significant stiffness degradation. At larger wall deflections, the walls exhibited stiffness degradation and pinched hysteretic load-deflection response. The walls incorporating 7.5 mm thick plywood sustained total wall deflections between 60 and 75 mm prior to failure. Wall ultimate strength occurred at lozenging deflections of 45 mm, and was between 1.4 to 2.6 times the design earthquake load $V_E$. The walls incorporating 12 mm thick plywood demonstrated improved stiffness and strength properties from the 7.5 mm thick plywood sheathed walls, sustaining total wall deflections of at least 90 mm prior to failure. The sheathing nailing was shown to comprise an ideal ductile component
exhibiting significant displacement capacity. Furthermore, the sheathing nailing permitted the shear wall to be easily inspected and repaired after a major earthquake attack.

2. Failure Modes

Testing was terminated after failure of the sheathing nails. For the nails driven through 7.5 mm thick plywood, failure occurred through the nail heads pulling through the sheathing, or the nail point pulling out of the framing. Very few nails failed through low cycle fatigue. For the nails driven through 12 mm thick plywood, failure was restricted to the nail points progressively withdrawing from the framing.

3. Framing Joint Behaviour

The framing joint forces arising from the nail force components acting in the direction perpendicular to the framing member exceeded the strength of the two 100 x 4.0 mm nails provided as the joint connection in Walls S1 and S2. This resulted in up to 8 mm separation of the framing joints. A nailed steel bracket connection recessed into the framing member under the sheathing was the only effective method of resisting the framing joint forces.

4. Nail Slip

Nail slip occurred in directions parallel and perpendicular to the framing members. Nail force components acting in the direction perpendicular to the framing stud caused the framing to bend. Good agreement exists between the predicted nail slip given by the proposed theory (Chapter 2), and the experimental points.
5. Load-Deflection Relationship

The wall lozenging load-deflection response was well predicted by the proposed elastic shear wall theory. Wall ultimate strength was estimated reasonably well by the proposed ultimate strength model.

6. Dynamic Load-Deflection Response

Wall subjected to dynamic loading exhibited enhanced stiffness and strength properties compared to walls subjected to quasi-static loading. The experimental load-deflection hysteresis loops were well modelled by the proposed pinching hysteretic model. Very good agreement was obtained between the experimental and theoretical dynamic response. Although the viscous damping model leads to a convenient form of the dynamic equations of motion, and adequately predicts the displacements, it does not predict the strength enhancement apparent in the dynamic tests.

7. Damping

Wall damping was approximately 10% during elastic response, (wall deflections less than 15 mm). After stiffness degradation had developed, the damping ratio used in the viscous damping model of the theoretical analysis was generally less than 5%. Overall damping, including hysteretic damping, determined from the free vibration decay, was in excess of 40%. Wall damping values during elastic response were observed to decrease with the age of the nailed sheathing joint, probably due to reduced friction forces at the joint interface between the sheathing and framing.

8. Shear Wall Age

Shear walls tested immediately after fabrication can be expected to exhibit enhanced stiffness, strength, and damping properties compared to those tested several months after fabrication.
6

THE SEISMIC RESPONSE OF STRUCTURES INCORPORATING PINCHED Hysteresis LOOPS

6.1 INTRODUCTION

Most structures designed to code recommended lateral forces can be expected to sustain several cycles of large inelastic deformations during a severe earthquake attack. The inelastic deformation demands may be evaluated by means of nonlinear time history analysis.

Early investigators [107] showed that the displacement demand $\Delta_{\text{max}}$ of classical elastoplastic single degree of freedom (SDOF) oscillator was strongly influenced by the initial natural period $T_o$, and was generally insensitive to the yield load $V_y$ (see Fig. 6.3). Consequently, the displacement demand $\Delta_{\text{max}}$ of such oscillators was very similar to that obtained for a linear elastic oscillator having identical initial natural period $T_o$; this has commonly been referred to as the equal displacement relationship, as shown in Fig. 6.1.
The equal displacement relationship may be explained with reference to the elastic response spectrum plotted in Fig. 6.2. The plastified regions of the elastoplastic oscillator reduced the effective stiffness from that of the linear elastic oscillator. The reduction caused an increase in period, which for an oscillator having an initial natural period greater than the period of the resonant peak, causes a reduction in the response acceleration. In addition, the plastified regions dissipate energy in the form of hysteretic damping. It is evident from the Fig. 6.2 response spectrum that an increase in period of oscillators having initial natural periods less than the period of the resonant peak may cause a substantial increase in the response acceleration. This may only be partially compensated by the increase in damping due to hysteretic damping, and the equal displacement assumption (Fig. 6.1) may be nonconservative for short period oscillators, as reported elsewhere [108, 109].
Fig. 6.2 Elastic Response Spectrum for a single degree of freedom linear oscillator.

The effect of stiffness degradation of reinforced concrete structures during reverse cycling was firstly investigated by Clough [110]. Clough compared the response of a SDOF elastoplastic oscillator with the response of the SDOF degrading oscillator shown in Fig. 6.3(b), and showed that for all but short initial natural periods $T_0$, the displacement demands were independent of the hysteretic idealisation used. Similar findings have been reported by others [111, 112, 113] for multi-degree of freedom (MDOF) structures. Thompson [114] investigated the response of the SDOF prestressed concrete idealisation shown in Fig. 6.3(c) for a range of initial natural periods $T_0$ and earthquake accelerograms. Thompson found that the displacement demand of the prestressed concrete idealisation was on average 1.3 times that of a reinforced concrete oscillator (represented by the Ramberg-Osgood idealisation) having the same yield load $V_y$, initial stiffness $k_0$ and damping ratio. However, this ratio of displacement demand varied significantly from the average value of 1.3 depending on the earthquake accelerogram and initial natural period $T_0$ used.
Fig. 6.3 Hysteresis Idealisations.
(a) Elastoplastic Idealisation.
(b) Clough's Degrading Stiffness Idealisation.
(c) Thompson's Prestressed Concrete Idealisation.
In this chapter, the effect of pinched hysteresis loops (characteristic of plywood sheathed shear walls) on the displacement demand of a structure is investigated. The approached followed is to compare the displacement demand of a SDOF elastoplastic oscillator with that of a SDOF pinching hysteretic oscillator.

6.2 NONLINEAR TIME HISTORY ANALYSIS

6.2.1 Equations of Motion

The dynamic response of a SDOF nonlinear oscillator having inertia mass $M$, secant stiffness $k(t)$ and viscous damping $c$ (see Fig. 6.4), and subjected to ground acceleration $\ddot{y}_g(t)$, satisfies the equation of motion,

$$M\dddot{y}(t) + c\dot{y}(t) + k(t)y(t) = -M\ddot{y}_g(t) \quad (6.1)$$

where $\dddot{y}(t), \dot{y}(t)$ and $y(t)$ are the response acceleration, velocity and displacement of the inertia mass $M$ relative to the ground at time $t$. Dividing Eq. 6.1 by $M$,

$$\dddot{y}(t) + 2\lambda\omega_{nat}\dot{y}(t) + \frac{k}{k_o}\omega_{nat}^2y(t) = -\ddot{y}_g(t) \quad (6.2)$$

where

$$\frac{k(t)}{m} = \frac{k(t)}{k_0} \times \frac{4\pi}{T_o^2} \quad (6.3)$$

$$\frac{c}{m} = \frac{4\pi\lambda}{T_o} \quad (6.4)$$

$$T_o = \frac{2\pi}{\omega_{nat}} = 2\pi \sqrt{\frac{M}{k_o}} = \text{initial natural period of vibration}$$

$$\lambda = \frac{c}{c_r} = \text{Ratio of critical damping}$$

$$c_r = 2\omega_{nat} M = \text{Critical damping coefficient}$$

$k_o = \text{elastic stiffness}$
6.2.2 Hysteresis Models

For each time step in the solution of the equation of motion (Eq. 6.2) the stiffness \( k(t) \) is determined from an algorithm idealising the load-deflection response of the nonlinear oscillator. A elastoplastic and pinching nonlinear load-deflection idealisations have been used in this study.

6.2.2.1 Elastoplastic Idealisation

The elastoplastic load-deflection hysteresis model is illustrated in Fig. 6.3(a). The initial loading path follows the initial stiffness \( k(t) = k_0 \) up to yield load \( V_y \), and then has a zero stiffness. Thereafter on unloading or reloading, the elastic stiffness is followed until the yield load \( V_y \) is reached in either loading direction.
Fig. 6.5 Pinching Hysteresis Idealisation of Plywood Sheathed Shear Walls.
6.2.2.2 Pinching Idealisation

The "pinching" hysteresis model developed to idealise the load-deflection behaviour of plywood sheathed shear walls is shown in Fig. 6.5(a). The initial loading path follows the initial stiffness $k(t) = k_0$ up to yield load $V_y$ and then reduced stiffnesses $k(t) = r_1 k_0$ and $k(t) = r_2 k_0$.

Unloading from the initial loading path and loading in the opposite direction results in stiffness degradation and pinching of the load-deflection hysteresis loops. The stiffness when unloading from the initial loading path, co-ordinates $(\Delta_{\text{max}}', V_{\text{max}}')$ is $k(t) = r_{\text{un}} k_0$. On the load reversal path, which commences when the load changes sign $(\Delta_0', 0)$, a degrading stiffness $k(t) = k_d$ is followed. The degrading stiffness $k_d$ is the slope of the line between the co-ordinates $(\Delta_0', 0)$ and the load offset co-ordinates $(0, V_{\text{os}}')$, as shown in Fig. 6.5(a). If the magnitude of the yield load - $V_y$ in this loading direction has not been exceeded, the loading path follows the degrading stiffness up to the intersection co-ordinates $(\Delta_{\text{ie}}', V_{\text{ie}}')$, and then follows the initial stiffness $k(t) = k_0$, $k(t) = r_1 k_0$ etc. If the magnitude of the yield load - $V_y$ has been exceeded, the loading path follows the degrading stiffness $k_d$ up to the intersection co-ordinates $(\Delta_{\text{ip}}', V_{\text{ip}}')$, and then follows the pinching stiffness $k(t) = k_p$, where

$$k_p = k_0 \left[ \frac{\Delta_{\text{ip}}'}{\Delta_{\text{un}}} \right]^\alpha$$

(6.5)

and

$$\Delta_{\text{un}} = \beta \Delta_{\text{max}}$$

(6.6)

$\Delta_{\text{max}}$ is the previous maximum deflection in the respective loading direction

$\Delta_y$ is the yield displacement

$\alpha$ is the "pinching" parameter, and controls the rate of stiffness degradation

and $\beta$ is a softening parameter.
The intersection co-ordinate \((\Delta_{ip}', V_{ip}')\) is positioned such that
the pinching loading path intersects with the intersection co-ordinate \((\Delta_{un}', V_{un}')\).

For partial load reversals, the loading path follows the existing degrading and pinching stiffnesses, as shown by the small inner loop in Fig. 6.5(a).

A special case occurs when the deflection intercept co-ordinate \((\Delta_{o}', 0)\) is small. In such cases, the degrading stiffness \(k_d\) (controlled by the load offset co-ordinate \((0, V_{os})\)) becomes large, and the intersection co-ordinate \((\Delta_{ie}', V_{ie})\) or \((\Delta_{ip}', V_{ip})\) does not exist. When this occurs, the degrading stiffness \(k_d\) is reduced, such that the loading path intersects with co-ordinates \((\Delta_{ie}', V_{ie} = 2 V_{os})\) or \((\Delta_{ip}', V_{ip} = 2 V_{os})\), as shown in Fig. 6.5(b).

A further option is included, whereby an initial slackness \(\Delta_{pgap}\) in the positive direction and \(\Delta_{ngap}\) in the negative direction is provided, as shown in Fig. 6.5(c). In this case, the initial loading path follows initial stiffness \(k(t) = 0\) until co-ordinates \((\Delta_{pgap}', 0)\) or \((\Delta_{ngap}', 0)\) and then follows the initial stiffness \(k(t) = k_o\) etc. as previously.

6.2.3 Time History Analysis

The computer program EQUAKE was developed to calculate the response of a nonlinear SDOF oscillator using the equation of motion (Eq. 6.2) together with the hysteresis models described above. Equation 6.2 was solved using a step-by-step numerical integration technique, as described by Clough and Penzien [95].

The integration was carried out using Newmark's constant average acceleration method [95] which is unconditionally stable for any time step \(\Delta_t\). A sensitivity analysis of the oscillators response showed that a time step of \(\Delta_t = 1/100\) seconds was adequate, and that smaller time steps \(\Delta_t = 1/200\) and \(1/400\) gave similar results. A fortran listing of the program EQUAKE is given in Appendix C.
6.3 COMPARISON OF ELASTOPLASTIC AND PINCHING HYSTERESIS RESPONSE

In order to compare the response of the pinching and elastoplastic models to a given earthquake ground motion record, the damping ratio $\lambda$, natural period of vibration $T_0 = 2\pi \sqrt{\frac{m}{k_0}}$ (and initial stiffness $k_0$), nominal yield load $\dot{V}_y$ were made identical for each comparison. The response of the two models were compared for oscillator natural periods $T_0$ between 0.1 and 2.0 sec. and having various damping ratios, nominal yield loads and subjected to different earthquake accelerograms, as discussed subsequently.

6.3.1 Earthquake Accelerograms

Four strong motion earthquake accelerograms were used in the following analyses:

(1) El Centro May 1940 NS
(2) Parkfield 1966 NSOE
(3) Pacoima Dam, San Fernando 1971 S14W
(4) Bucharest 1977 NS

The four earthquake accelerograms were selected to provide a range of earthquake characteristics. Only the first 10 seconds of each earthquake record was used.

The El Centro accelerogram was the first strong motion accelerogram to be recorded. It has been widely used to investigate the seismic response of structures, and forms the basis of many of the early seismic loading code requirements. The El Centro accelerogram contains a series of major pulses each with different amplitude and frequency characteristics.

The Parkfield accelerogram was recorded near the epicentre, and essentially has only one major acceleration pulse.

The Bucharest accelerogram was recorded 160 km from the epicentre after the seismic waves had propagated through alluvial deposits,
and contains higher acceleration pulses in the long period range compared to the other earthquake accelerograms.

The Pacomia Dam accelerogram was one of the most severe earthquake records available at the time of this study. Due to local site effects, this record contains high accelerations in the short period range.

Response spectra for the four earthquakes are presented in Fig. 6.6.

![Elastic Response Spectrum](image)

**Fig. 6.6** Elastic Response Spectrum for a single degree of freedom linear oscillator to El Centro 1940, Parkfield 1966, Pacoima Dam 1971, and Bucharest 1977 Accelerograms.
6.3.2 **Damping Ratio** $\lambda$

Results from the shakeable tests reported in Chapter 5 indicate that the damping ratio for plywood sheathed shear walls lies between 10% and 20% during elastic response, but reduces to a value as low as 2% during post-elastic response. However, the appropriate damping ratio for a building which incorporates shear walls may be significantly higher than the values obtained for isolated walls, due to the effects of non-structural elements in buildings, such as partitioning, etc.

In the following analyses, damping ratios of 2% and 10% were used to investigate the sensitivity of damping ratio on the response of the oscillator.

6.3.3 **Nominal Yield Load** $F_y$ and **Parameters Controlling Hysteresis Models**

To compensate for the varying strengths of the four earthquake accelerograms, the nominal yield load of both models (Figs. 6.3 and 6.5) was defined as

$$V_y = R V_e$$

where $V_e$ is the maximum base shear developed in an elastic oscillator having the same initial natural period of vibration $T_0$, and subjected to the same ground motion record as the nonlinear oscillator, and $R$ is the force reduction factor. The response of the two models were compared for force reduction values of $R = 1/2, 1/4$ and $1/8$. 
Table 6.1 Control Parameters for Pinching Hysteresis Idealisation.

<table>
<thead>
<tr>
<th>Control Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V'$</td>
<td>1.5 $V_y$</td>
</tr>
<tr>
<td>$V_{os}$</td>
<td>0.25 $V_y$</td>
</tr>
<tr>
<td>$r_1$</td>
<td>0.34</td>
</tr>
<tr>
<td>$r_2$</td>
<td>0.00</td>
</tr>
<tr>
<td>$r_{un}$</td>
<td>1.45</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>0.38</td>
</tr>
<tr>
<td>$\beta$</td>
<td>1.09</td>
</tr>
<tr>
<td>$\Delta_{pgap}$</td>
<td>0.00</td>
</tr>
<tr>
<td>$\Delta_{ngap}$</td>
<td>0.00</td>
</tr>
</tbody>
</table>

For consistency between the elastoplastic and pinching models, the nominal yield load $V_y$ has been conveniently defined as the design earthquake load $V_E$. Consequently, to be consistent with the test walls described in Chapter 5, the ultimate strength $V' = 1.5 V_E = 1.5 V_y$.

Table 6.1 tabulates the control parameters in terms of $V_y$ for the pinching model. The parameters tabulated result in pinching hysteresis loops that are similar to those determined from the tests reported in Chapter 5.
Fig. 6.7 Predicted Load vs Displacement Response of Pinching and Elastoplastic Hysteretic Idealisations to the El Centro 1940 Accelerogram for damping ratio $\lambda = 10\%$, initial natural period $T_o = 0.6$ sec, and Force Reduction $R = 0.250$.

6.3.4 Results and Discussion of Hysteretic Response

Figure 6.7 shows the actual load-deflection hysteretic record of both models for $T_o = 0.6$ sec, $\lambda = 10\%$ and $R = 0.250$, and subjected to the first 10 seconds of the El Centro accelerogram. Similar records were generated for other initial natural periods $T_o$ and accelerograms. The predicted displacement demands of both hysteresis models subjected to El Centro, Parkfield and Pacoima Dam are plotted in Fig. 6.8 against initial natural period $T_o$. Displacement demand $\Delta_{max}$ is expressed here in terms of
displacement ductility demand $\mu_A = \Delta_{\text{max}} / \Delta_y$ where $\Delta_y = V_y / k_o$ is the yield displacement. The yield displacement is identical for the elastoplastic and pinching models, since they have the same yield load $V_y$ and initial stiffness $k_o$.

Fig. 6.8 Displacement Ductility Demands on Degrading and Elastoplastic Idealisations to El Centro 1940, Parkfield 1966, and Pacoima Dam 1971 Accelerograms.
The ductility ratio $r_\mu$ is introduced and defined as

$$r_\mu = \frac{\mu_{\Delta, P}}{\mu_{\Delta, EP}}$$

where $\mu_{\Delta, P}$ is the displacement ductility demand of the pinching model and $\mu_{\Delta, EP}$ is the displacement ductility demand of the elastoplastic model at a specified initial natural period $T_0$. The ductility ratios $r_\mu$ for a range of initial natural periods $T_0$

![Graph showing the relationship between Ductility Ratio $R$ and Period $T_0$ for different values of $R$. The graph includes lines for Bucharest, Pacoima, El Centro 1940, and Parkfield with a force reduction factor $\lambda = 10\%$.]

**Fig. 6.9** Displacement Ductility Ratio $r_\mu$ plotted against initial natural period $T_0$ and Force Reduction Factor $R$. 
The average ductility ratio $r^\mu$ for all four earthquake accelerograms is plotted in Fig. 6.10 against initial natural period $T_o$ and for values of the force reduction factor of $R = 0.500$, $0.250$, and $0.125$ and damping ratio of $\lambda = 2\%$ and $10\%$. Although the average ductility demand for the pinching model was greater than that for the elastoplastic model for some combinations of damping ratio, natural period and force reduction factor, differences were generally less than $20\%$. Overall, the ductility ratio is not influenced by the force reduction factor $R$ or the value of damping ratio $\lambda$ used in the analysis. The average ductility ratio was $r^\mu = 0.96$, indicating that the displacement demands of the pinching model are similar to the elastoplastic model. The linear regression of ductility ratio $r^\mu$ with initial natural period $T_o$ shown in Fig. 6.10 shows that $r^\mu$ increases slightly with natural period $T_o$.

The area enclosed by the load-deflection hysteresis loops during a cycle represents the hysteretic energy dissipation of the oscillator, and is referred to here as hysteretic damping. The hysteretic damping at a specified displacement ductility factor $y_\Delta$ of the pinching model is plotted in Fig. 6.11 as a ratio of that of the elastoplastic model. Curve 1(a) shows that the ratio of hysteretic damping before stiffness degradation i.e. during loading along the parent monotonic loading curve is between $50\%$ and $60\%$. Stiffness degradation after repeated loading to a specified displacement ductility (curve 1(b)) causes this ratio to reduce to between $30\%$ and $40\%$. Fig. 6.11 also shows that the hysteretic damping ratio before and after stiffness degradation reduces with increasing displacement ductility. The results from
this study indicate that such reductions in hysteretic damping have little influence on the total displacement demand.

**Fig. 6.10** Average Displacement Ductility Ratio $\mu_r$ plotted against initial natural period $T_0$, Force Reduction Factor $R$ and Damping Ratio $\lambda$. 
Fig. 6.11 Energy Dissipation within Hysteretic loop of pinching idealisation with and without strength reserve.

The reason for this may be explained with reference to the Fig. 6.2 response spectrum. Figure 6.2 shows that the response of the oscillator decreases with increasing period, and as the period moves away from the resonant peak the response is relatively insensitive to damping. The natural period $T_o$ of a slackness oscillator (Fig. 6.16(a)) representing the pinching hysteresis loop, from Section 5.4.3 is

$$T_o = T_s \left(1 + \frac{2 A_s}{\pi A - A_s}\right) \quad (6.7)$$

where $T_s = 2\pi \sqrt{m/k_s}$ is the natural period of the oscillator without slackness $A_s$. Equation 6.7 shows that the natural period of the slackness oscillator increases as the spring stiffness $k_s$ decreases, and as the slackness $A_s$ increases.

Table 6.2 Variation in Control Parameters for Pinching Hysteresis Idealisation.

<table>
<thead>
<tr>
<th>Control Parameter</th>
<th>Original Parameter</th>
<th>Parameter Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$v'$</td>
<td>$v'$</td>
<td>$v'$</td>
</tr>
<tr>
<td>$v_{os}$</td>
<td>0.25 $v'$</td>
<td>0.25 $v'$</td>
</tr>
<tr>
<td>$r_1$</td>
<td>0.34</td>
<td>0.34</td>
</tr>
<tr>
<td>$r_2$</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$r_{un}$</td>
<td>1.45</td>
<td>1.45</td>
</tr>
<tr>
<td>$a$</td>
<td>0.38</td>
<td>0.38</td>
</tr>
<tr>
<td>$\beta$</td>
<td>1.09</td>
<td>1.09</td>
</tr>
<tr>
<td>$k_{gap}$</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$\phi_{gap}$</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
6.4 SENSITIVITY OF THE RESPONSE OF THE PINCHING MODEL TO CHANGES IN CONTROL PARAMETERS

The influence of the control parameters of the pinching model on its seismic response is investigated in this section. The investigation was limited to the El Centro accelerogram and a damping ratio $\lambda = 10\%$. Table 6.2 summarises the control parameters that were varied.

![Graph showing force vs displacement for different parameters]

**Fig. 6.12** Predicted Load vs Displacement Response of Pinching Idealisation with and without strength reserve to the El Centro 1940 Accelerogram for damping ratio $\lambda = 10\%$, initial natural period $T_0 = 0.6$ sec, and Force Reduction Factor $R = 0.250$. 
6.4.1 Results and Discussion of Hysteretic Response

6.4.1.1 Effect of Strength Reserve

The effect of the strength reserve $1.5V_y$ of the pinching model on the displacement demands is investigated in this section. The original pinching model was modified so that loads exceeding $V_y$ could not be sustained. Figure 6.12 shows the actual load-deflection hysteretic record of the original pinching model and the modified pinching model (Fig. 6.5: $V' = V_y$) for an initial natural period $T_o = 0.6$ sec., damping ratio $\lambda = 10\%$ during the El Centro accelerogram. The displacement demand for the modified pinching model (i.e., without load reserve) is only 10% greater than that for the original pinching model. The predicted displacement ductility demands for a range of initial natural periods $T_o$ are collated in Fig. 6.13 for the original and modified pinching models and for the elastoplastic model. The modified ductility ratio $r_{u,m}$ is introduced and is defined as

$$r_{u,m} = \frac{\mu_{u,m}}{\mu_{u,P}}$$

where $\mu_{u,m}$ is the displacement ductility demand of the modified pinching model, and $\mu_{u,P}$ is the displacement ductility demand of the original pinching model. The modified ductility ratio $r_{u,m}$ for a range of initial natural periods $T_o$ and reduction factors $R$ is collated in Fig. 6.14. For the El Centro accelerogram the displacement demands for the modified pinching model are an average 10% greater than those for the original pinching model, and 15% greater than those for the elastoplastic model, as shown in Table 6.3.
Table 6.3 Summary of Results from Sensitivity Analysis.

<table>
<thead>
<tr>
<th>Parameter Variation</th>
<th>$r_{um} = \frac{\Delta_{A,PM}}{\Delta_{A,p}}$</th>
<th>$r_u = \frac{\Delta_{I,PM}}{\Delta_{I,EP}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Force reduction factor $R$</td>
<td>Force reduction factor $K$</td>
</tr>
<tr>
<td>---------------------</td>
<td>-----------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>1 $\nu' = \nu_y$</td>
<td>1.12 1.15 1.03 1.10 1.01 1.22 1.22 1.15</td>
<td>1.12 1.15 1.03 1.10 1.01 1.22 1.22 1.15</td>
</tr>
<tr>
<td>2 $\alpha = 0.0$</td>
<td>1.05 1.06 1.06 1.06 0.95 1.12 1.26 1.11</td>
<td>1.05 1.06 1.06 1.06 0.95 1.12 1.26 1.11</td>
</tr>
<tr>
<td>3 $\alpha = 1.0$</td>
<td>0.93 0.89 0.92 0.91 0.84 0.94 1.09 0.96</td>
<td>0.93 0.89 0.92 0.91 0.84 0.94 1.09 0.96</td>
</tr>
<tr>
<td>4 $r_{un} = 5.0$</td>
<td>0.95 0.91 0.90 0.92 0.86 0.96 1.07 0.96</td>
<td>0.95 0.91 0.90 0.92 0.86 0.96 1.07 0.96</td>
</tr>
<tr>
<td>5 $v_{os} = 0.0$</td>
<td>1.36 1.35 1.19 1.30 1.22 1.43 1.41 1.35</td>
<td>1.36 1.35 1.19 1.30 1.22 1.43 1.41 1.35</td>
</tr>
<tr>
<td>6 $\Delta_{Pgap} = \Delta_{Hgap} = 10$ mm</td>
<td>0.76 1.05 1.02 0.94 0.88 0.96 0.91 1.00</td>
<td>0.76 1.05 1.02 0.94 0.88 0.96 0.91 1.00</td>
</tr>
<tr>
<td>7 $\beta = 1$</td>
<td>0.98 0.97 0.97 0.97 0.88 1.03 1.15 1.02</td>
<td>0.98 0.97 0.97 0.97 0.88 1.03 1.15 1.02</td>
</tr>
</tbody>
</table>

$\lambda = 10\%$ during El Centro 1940 NS.

Fig. 6.13 Displacement Ductility Demands on Elastoplastic, Pinching Idealisations with and without strength reserve to the El Centro 1940 Accelerogram for damping ratio $\lambda = 10\%$, and Force Reduction Factor $R = 0.250$. 
Fig. 6.14 Modified Ductility Ratio $r_{du}$ plotted against initial natural period $T_0$ and Force Reduction Factor $R$ for pinching Idealisation without strength reserve.

The hysteretic damping of the modified pinching model shown in Fig. 6.11 as a ratio of hysteretic damping of the elastoplastic model over a range of displacement ductility factor $u_\Delta$. The hysteretic damping of the modified pinching model is significantly less than the original pinching model during the initial loading cycle to each displacement, but is very similar to the original pinching model during repeated loading cycles to each displacement.

The small effect that the strength reserve has on the response of the pinching oscillator is consistent with the equal displacement relationship (see Fig. 6.1), which indicates that the displacement demands are insensitive to the oscillator strength.
Fig. 6.15 Average Modified Ductility Ratio $\mu_m$ plotted against initial natural period $T_0$ for parameter variations 2 to 5.

6.4.1.2 Effect of Varying Pinching, Unloading and Degrading Stiffness

The analyses reported previously were all made using pinching, unloading and degrading stiffnesses similar to those found during experimental tests of plywood sheathed shear walls. In order to assess the sensitivity of the displacement ductility demand of the pinching, unloading and degrading stiffnesses, each of the latter was varied independently of the others, as summarised in Table 6.2: Parameter variations 2 to 5.

The average value of the modified ductility ratio $\mu_m$ is calculated from the modified ductility ratios $\mu_m$ determined for
the force reduction factors $R = 0.5$, $0.25$ and $0.125$. The average modified ductility ratio is plotted in Fig. 6.15 against the initial period $T_0$ for each parameter variation. Figure 6.15 shows that the effect of each variable on the displacement demand is greatest for short initial natural periods, but becomes insignificant for longer initial natural periods.

(a) **Variations in the Pinching Stiffness: $\alpha = 0.0$ and $\alpha = 1.0$**

(Table 6.2: Parameter Variation 2 and 3)

The parameter value $\alpha = 0$ constrains the pinching stiffness to be equal to the initial stiffness $k_0$ irrespective of the maximum displacement attained (Fig. 6.5). This results in an increase in the overall average displacement demand of 6% (see Fig. 6.15 and Table 6.3). The parameter value $\alpha = 1.0$ results in significantly increased degradation of the pinching stiffness from the original pinching model. The increased degradation of the pinching stiffness causes an overall average decrease in displacement demand of approximately 10% (see Table 6.3).

The results for the El Centro accelerogram record indicate that degradation of the pinching stiffness leads to a reduction in the overall displacement demands.

(b) **Variation to the Unloading Stiffness: $r_{\text{un}} = 5.0$**

(Table 6.2: Parameter variation 4)

The effect of increasing the unloading stiffness from that of the original pinching model decreased the overall average displacement demands by 8% (see Table 6.3). The effect of increased unloading stiffness on the response of the oscillator may be explained with reference to the SDOF slackness oscillator shown in Fig. 6.16. For an oscillator having an unloading stiffness equal to the loading stiffness, as represented in Fig. 6.16(a), the area under the load-deflection curve (Fig. 6.16(a): area abc)
represents the potential energy stored at maximum deflection $\Delta_{\text{max}}$. As the mass returns to the slackness region, this energy is converted to kinetic energy. If the unloading stiffness is several times the loading stiffness as shown in Fig. 6.16(b), the potential energy stored at maximum deflection (Fig. 6.16(b): area abc) is the same as before, but the energy converted to kinetic energy is represented by the significantly reduced area (Fig. 6.16(b): area bcd). Consequently, less of the potential energy is returned as kinetic energy to the oscillator when the unloading stiffness is greater than the loading stiffness. The low unloading stiffness of the Thompson [114] prestressed concrete idealisation (Fig. 6.3(c)) is believed to be the reason for the 30% increase in displacement demand reported by Thompson [114] for this particular hysteretic loop shape.

![Diagrams](image)

**Fig. 6.16** Response of slackness oscillators to Earthquake motions. (a) Slackness Oscillator. (b) Slackness oscillator with increased unloading stiffness.
(c) **Variation to the Degrading Stiffness**: \( V_{os} = 0.0 \)

(Table 6.2: Parameter variation 5)

The parameter value \( V_{os} = 0.0 \) resulted in the degrading stiffness \( k_d \) (Fig. 6.5) reducing to zero. Zero degrading stiffness caused an overall average increase in displacement demand of 30% for the El Centro accelerogram (see Fig. 6.15 and Table 6.3). It appears that the displacement demand for the pinching model is strongly influenced by the load \( V_{os} \) sustained within the slackness region, even though it is small compared to the nominal yield load \( V_y \). Further studies are required to determine whether this effect of \( V_{os} \) on the displacement demand is only a characteristic of the El Centro accelerogram or whether it is a characteristic of all earthquake accelerograms.

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**Fig. 6.17** Modified Ductility Ratio \( \mu_{im} \) plotted against initial natural period \( T_o \) and Force Reduction Factor \( R \) for pinching Idealisation with 10 mm initial slackness.
6.4.1.3 Effect of Initial Slackness

The effect of including $A_{P_{gap}} = A_{N_{gap}} = 10$ mm initial slackness in the original pinching model is shown in Fig. 6.17. Results show an increase in displacement demand for initial natural periods $T_o$ less than about 0.5 seconds and a decrease in displacement demand for initial natural periods $T_o$ greater than about 0.5 seconds. The slackness caused an overall average reduction in displacement demand of 6%, as shown in Table 6.3. Equation 6.7 shows that the natural period of the slackness oscillator representing the pinching hysteresis loops increases with increasing slackness $A_s$. The influence of increased natural period can be seen with reference to Fig. 6.2 where the base shear demand can be expected to increase for spring natural periods $T_s$ less than the period at the resonant peak and decrease for $T_s$ greater than the period at the resonant peak.

6.4.1.4 Effect of Softening Parameter

Restraining the softening parameter $\beta$ to unity causes an overall average reduction in displacement demand of 3%, as shown in Table 6.3.

6.5 SUMMARY OF MAJOR FINDINGS

The displacement demands on the pinching oscillator are on average very similar to those on the elastoplastic oscillator, despite having 50% less hysteretic damping. It appears that the primary mechanism limiting the displacement demands of nonlinear oscillators is the increase in effective period, which itself depends on the incremental stiffness of the hysteretic model.
7

SUMMARY AND RECOMMENDATIONS FOR FUTURE RESEARCH

7.1 SUMMARY

A design methodology for earthquake resistant plywood sheathed shear walls is presented, based on the capacity design philosophy [9]. In this, the sheathing nailing is selected as the preferred ductile component, and the framing members, sheathing, and anchorage connections are then designed to resist the overstrength forces imposed by the sheathing nailing. Consequently, brittle shear, bending, and tension failure of the timber members or anchorage connections are prevented. In order to develop the design procedure, elastic and ultimate strength shear wall models were formulated. They show that sheathing nail force components acting in the direction perpendicular to the edge of the sheathing induce framing joint forces and framing flexural stresses, both of which are neglected by the present code [11] design expressions. Four full scale plywood sheathed shear wall units were subjected to quasi-static reverse cyclic loads with the primary aim of validating the proposed shear wall models. The test walls were comprehensively instrumented with linear potentiometers measuring nail slip in the directions
parallel and perpendicular to the sheathing edge. Very good agreement was obtained between the experimental and the theoretical behaviour.

The experimental tests verified the shear wall models, which showed that wall strength and stiffness were primarily governed by the sheathing nailing. Furthermore, the sheathing nailing was shown to be an ideal ductile component, exhibiting significant displacement capacity while maintaining strength. Consequently, the sheathing nail behaviour was investigated in some detail with a series of nailed sheathed joints, subjected to quasi-static reverse cyclic loads. The results from the experimental study demonstrated that more stable ductile joint behaviour occurred with nail point withdrawal compared to nail head pull through. An ultimate strength model for nailed joints was developed to predict 5\%ile, 50\%ile and 95\%ile characteristic strengths. The model is applicable to nailed joints in which the nail point withdraws from the timber. Excellent agreement was obtained between the experimental and theoretical ultimate strengths of nailed sheathed joints. The ultimate strength model identified nail diameter, nail length and nail coating as being the most influential parameters affecting joint strength. An expression for the overstrength factor $\phi_o$ of nailed sheathed joints is proposed by which the overstrength of the joints above their respective design earthquake load $P_E$ may be predicted. The overstrength factor $\phi_o$ depends on the nail diameter, nail length, nail coating, timber moisture content, and loading rate.

A theoretical time-history nonlinear single degree of freedom dynamic computer program was developed to predict the response of plywood sheathed shear walls to ground excitation. Complementary to the computer program, seven full scale plywood sheathed shear wall units were subjected to either sinusoidal or earthquake shaketable excitation. The primary aim of the dynamic tests was to validate the proposed dynamic model and determine the magnitude of damping to be used in the computer idealisation.
The results from the shaketable tests showed:

(a) The experimental load-deflection hysteresis loops were accurately modelled by the proposed dynamic pinching idealisation.

(b) Walls subjected to dynamic loading exhibited enhanced stiffness and strength properties compared to walls subjected to quasi-static loading.

(c) The damping ratio to be used in the dynamic analysis was approximately 10% during elastic response, but was less than 5% after stiffness degradation had developed.

(d) Wall elastic damping, stiffness, and strength decreased with the age of the sheathing nailing.

The effect of pinching in the hysteresis loops on the seismic response of plywood sheathed shear walls was demonstrated by comparing the response of a pinching single degree of freedom idealisation with that of an elastoplastic single degree of freedom idealisation. Comparisons were made for a range of initial natural periods, damping ratios, yield loads and earthquake accelerograms. The displacement demands for the pinching idealisation were on average very similar to those of the elastoplastic idealisations. It appears that the increase in effective natural period as large displacements develop in the inelastic range is the primary mechanism controlling the displacement demands of nonlinear oscillators.

7.2 RECOMMENDATIONS FOR FUTURE RESEARCH

The following recommendations for future research should lead to an improved seismic design methodology for plywood sheathed shear walls.

1. In the proposed seismic design methodology, the upper bound value for the sheathing nailing overstrength is believed to provide conservative protection against brittle failure of the non-ductile components. Further shear wall tests should be conducted to assess and improve the proposed design procedure.
2. The limited experimental study of nailed sheathing joints indicated that improved ductile joint behaviour can be expected when the nail point withdraws from the framing. Further experimental studies investigating the reverse cyclic behaviour of nailed sheathing joints are required in order to determine the types of nails and sheathing combinations that will enforce nail point withdrawal.

3. The proposed ultimate strength nailed joint model showed that the strength of a nailed sheathing joint was highly influenced by the withdrawal resistance of the nail shank. This, in turn, was shown to depend on the nail diameter, nail length, nail coating, change in timber moisture content, joint age, and loading rate. Dynamic tests of nailed sheathing joints are required to determine their ultimate strength. The test joints should be fabricated and left to curve in a similar manner to an actual shear wall to allow for the effects of joint age and cyclic moisture content changes. Tests are needed to identify the importance of nail type alone, including variations in nail diameter, nail length, and nail coating. An investigation into the effects of corrosion of bright steel nails on the ultimate strength of nailed sheathed joints is also required.

4. An aspect of the dynamic response of shear walls requiring additional research is their performance in multi-story buildings. The pinching hysteretic model developed in this study is suitable for incorporating into a more general multi-story time-history structural analysis programs. The effects of higher modes of vibration, and torsion loading may induce shear wall displacement demands greater than those indicated by the single degree of freedom analysis.
APPENDIX A1

DERIVATION OF STRESS BLOCK FACTORS $\alpha$, $\beta$ AND THE NORMAL BEARING LOAD $N_b$

In this Appendix the Stress Block factors $\alpha$, $\beta$ and the normal bearing load $N_b$ are derived.

A1 Derivation of Stress Block Factors $\alpha$, $\beta$

Figure A1 shows an embedded nail of diameter $d_n$ subjected to imposed normal load $N_b$. Several assumptions are made in the following derivation of the stress block factors:

1. The reaction stresses of the supporting timber fibres are related to the embedment of the nail shank at that location through a specified stress-embedment relationship, i.e. the timber stress at any location is independent of the stress adjacent to that location.

2. The nail penetration into the timber is sufficient for a plastic hinge to develop at position $z$ from the surface.

3. No embedment of the nail shank occurs between the nail point and the plastic hinge. Consequently, no bearing stresses act on the nail shank within this region.

4. Because no bearing stresses act between the nail point and the plastic hinge, shear force in the nail shank at the hinge position is assumed to be zero.

5. The nail remains straight except at the plastic hinge position.

The nail deflection $\Delta_{bs}$ at the timber surface arises entirely from rotation at the plastic hinge, and with reference to Fig. A1 for notation, is

$$\Delta_{bs} = \theta z$$  \hspace{1cm} (A1)

The actual bearing stresses on the nail shank are shown in Fig. A1, having a resultant
\[ N_b = \int_{0}^{z} d_n f_b \, dx \quad (A2) \]

where \( f_b \) is the bearing stress at location \( x \) along the nail shank. Given that the nail shank remains straight outside the plastic hinge position,

\[ \frac{\Delta_{bs}}{Z} = \frac{\Delta_b}{x} \quad (A3) \]

where \( \Delta_b \) is the nail embedment at location \( x \). For constant nail diameter \( d_n \) along the nail length, and substituting Eq. A3 into A2,

\[ N_b = \frac{d_n}{\Delta_{bs}} \int_{0}^{\Delta_{bs}} f_b \, d\Delta_b \quad (A4) \]

Moment equilibrium about the plastic hinge position (Fig. A1) requires the position \( x \) of the resultant of the actual bearing stress distribution to be

\[ -x = \frac{\int_{0}^{z} d_n f_b x \, dx}{\int_{0}^{z} d_n f_b \, dx} \quad (A5) \]

and from the Eq. A3 relationship,

\[ \frac{d_n Z^2}{\Delta_{bs}^2} \left[ \frac{\Delta_{bs}}{f_b \Delta_b} \right] \quad (A6) \]

It is helpful to replace the actual nonlinear bearing stress distribution acting on the nail shank with an equivalent rectangular stress block distribution incorporating stress block factors \( \alpha \) and \( \beta \), as shown in Fig. A1. The stress block factors themselves are determined from the conditions that the area and
centroidal position of the assumed rectangular stress block distribution are equal to those of the actual bearing stress distribution. The stress resultant \( N_b \) of the rectangular stress block distribution is

\[
N_b = a_b d_f Z f_b^n \tag{A7}
\]

where \( f_b \) is the bearing strength of the timber. For the magnitude of the stress resultant to be equivalent to the actual, then from Eqs. A4 and A7

\[
\alpha \beta = \frac{1}{f_b} \frac{\Delta_{bs}}{\Delta_b} \left[ \int_{0}^{\Delta_b} f_b d\Delta_b \right] \tag{A8}
\]

Moment equilibrium about the plastic hinge position requires the position \( x \) of the resultant of the rectangular stress block to be

\[
x = Z(1 - \sqrt{2}) \tag{A9}
\]

For equivalent stress resultant position, Eqs. A6 and A9

\[
(1 - \alpha \beta) = \frac{\Delta_{bs}}{f_b \Delta_b} \int_{0}^{\Delta_b} f_b d\Delta_b \tag{A10}
\]

The bearing stress-embedment properties of the timber can be well modelled in exponential form (see Section 3.4.1) by

\[
f_b = K_b \left( \Delta_o + \psi \Delta_b \right) \left( 1 - e^{\Delta_b/\Delta_o} \right) \tag{A11}
\]

where \( \Delta_o = f_b/k_b \) is the yield embedment and \( \psi \) is the bilinear factor.

The stress block factor product \( \alpha \beta \) in terms of this bearing stress-embedment relationship (Eq. A11), with reference to Eq. A8, is

\[
\alpha \beta = \frac{1}{k_b a_d \Delta_{bs}} \left[ k_b (\Delta_o + \psi \Delta_b) (1 - e^{-(\Delta_b/\Delta_o)}) d\Delta_b \right] \tag{A12}
\]
and introducing the ratio $\Delta_r = \frac{\Delta_{bs}}{\Delta_c}$,

$$\alpha = \frac{1}{\Delta_x} \left( (1 + \psi(\Delta_x + 1)) (e^{-\Delta_x} - 1) + \Delta_x (1 + \psi) + \frac{1}{2} \psi \Delta_x^2 \right)$$  \hspace{1cm} (A13)

and from Eqs. A10, A11 and A13,

$$\beta = \frac{2 \left( ((1 + \psi)(1 + \Delta_x) + \psi \Delta_x^2) (e^{-\Delta_x} - 1) + \Delta_x \left( \frac{1}{2} + \psi \right) (\Delta_x + 2) + \frac{1}{3} \psi \Delta_x^3 \right)}{\Delta_x \left( (1 + \psi(\Delta_x + 1)) (e^{-\Delta_x} - 1) + \Delta_x (1 + \psi) + \frac{1}{2} \psi \Delta_x^2 \right)}$$  \hspace{1cm} (A14)

The stress block factors $\alpha$ and $\beta$ are plotted in Fig. A2 against $\Delta_r$ and bilinear factor $\psi$.

**A2 Normal Bearing Load $N_b$**

Only failure modes III and IV (see Fig. 3.4) are considered applicable to nailed sheathed joints, as it is unlikely that failure modes I and II would develop because of the minimum nail penetration and minimum sheathing thickness limitations recommended by codes and design manuals [11, 49, 50].

(a) Mode III

Figure A3 shows a nailed sheathed joint where the sheathing is of sufficient thickness to cause the nail to deform in mode III. Equilibrium of the stress resultants acting on the nail shank shown in Fig. A3 requires

$$N_{b1} - N_{b2} = 0$$  \hspace{1cm} (A15)

where

$$N_{b1} = \beta_1 Z_1 d_n \alpha_1 f'_{b1}$$  \hspace{1cm} (A16)

$$N_{b2} = \beta_2 Z_2 d_n \alpha_2 \lambda f'_{b2}$$  \hspace{1cm} (A17)

$$\lambda = \frac{f'_{b2}}{f'_{b1}}$$

and $\alpha_1$, $\beta_1$, and $\alpha_2$, $\beta_2$ are the stress block factors for the framing (member 1) and sheathing (member 2) respectively. Solving Eq. A15 for $Z_1$ gives
\[ Z_1 = \frac{\beta_2 z_2 \lambda \alpha_2}{\beta_1 \alpha_1} \]  
\[ \text{(A18)} \]

Moment equilibrium about the plastic hinge position located in member 1 requires,

\[ \frac{M}{p} + N_{b1} e_1 - N_{b2} e_2 + M = 0 \]  
\[ \text{(A19)} \]

where \( M_p \) is the plastic moment of the nail and \( e_1 \) and \( e_2 \) are the distances from the plastic hinge to the centroid of each stress block, and are

\[ e_1 = Z_1 \left( 1 - \frac{1}{2} \beta_1 \right) \]  
\[ \text{(A20)} \]

\[ e_2 = Z_1 + \frac{1}{2} \beta_2 z_2 \]  
\[ \text{(A21)} \]

Substituting Eqs. A16 to A21 into Eq. A19 we have

\[ \frac{2M}{f'_{b1} d_{bn}} + \beta_1 Z_1^2 \alpha_1 \left( 1 - \frac{1}{2} \beta_1 \right) - \beta_2 z_2 \alpha_2 \lambda \left( Z_1 + \frac{1}{2} \beta_2 z_2 \right) = 0 \]  
\[ \text{(A22)} \]

Eliminating \( Z_1 \) and solving for \( Z_2 \) we find

\[ Z_2 = \sqrt{\frac{4M}{f'_{b1} d_{bn}} \left( \frac{1}{\beta_1^2 \alpha_2^2 \lambda^2 \alpha_1^2 + 1} \right)} \]  
\[ \text{(A23)} \]

Finally, combining Eqs. A17 with A23, the joint bearing load \( p_b \) can be written as

\[ p_b = \sqrt{\frac{4M f'_{b1} d \lambda \alpha_2}{\alpha_1 \lambda \alpha_2 + 1}} \]  
\[ \text{(A24)} \]

The stress block factors \( \alpha_1 \) and \( \alpha_2 \) in Eq. A24 depend on the bilinear factors \( \psi_1 \) and \( \psi_2 \), the yield embeddings \( \Delta_{01} \) and \( \Delta_{02} \) and the dowell embedment at the joint interface \( \Delta_{bs1} \) and \( \Delta_{bs2} \) in members 1 and 2 respectively (Eqs. A13 and A14). The magnitudes of \( \Delta_{bs1} \) and \( \Delta_{bs2} \) are related to joint slip \( \Delta_j \) by
\[ \Delta_{bs1} + \Delta_{bs2} = \Delta_j \]  
(A25)

From the geometry of the bent nail shank and assuming the nail shank remains straight between plastic hinge positions (Fig. A3),

\[ \frac{\Delta_{bs1}}{Z_1} = \frac{\Delta_{bs2}}{Z_2} = \tan \theta \]  
(A26)

and

\[ \Delta_j = (Z_1 + Z_2) \tan \theta \]  
(A27)

The normal bearing load \( N_b = N_{b1} = N_{b2} \) can be found for a specified joint slip \( \Delta_j \) and for known timber bearing-stress embedment characteristics. A trial and error calculation procedure is required to determine the normal bearing load \( N_b \). A general iterative procedure for finding \( Z_1 \) and \( Z_2 \) and hence \( N_b \) from Eq. A16) for a specified joint slip \( \Delta_j \) is summarised in Fig. A4. The procedure converges rapidly (within 2 or 3 iterations) for nailed sheathed joints at slips greater than 2mm.

(b) Mode IV

Figure A5 shows a nailed joint where the ratio of nail diameter to sheathing thickness is sufficiently large for failure mode IV to develop.

Using the notation shown in Fig. A4, equilibrium requires

\[ N_{b1} - N_{b2} + N_{b2} = 0 \]  
(A28)

where

\[ N_{b1} = \beta_1 Z_2 \alpha_1 f_{b1} d_n \]  
(A29)

\[ N_{b2} = \beta_2 Z_2 \alpha_2 \lambda f_{b1} d_n \]  
(A30)

\[ N'_{b2} = \beta_3 (\tau_2 - Z_2) \alpha_3 \lambda f_{b1} d_n \]  
(A31)

solving for \( Z_1 \), we find

\[ Z_1 = \frac{\lambda}{\alpha_1 \beta_1} \left[ Z_2 (\alpha_2 \beta_2 + \alpha_3 \beta_3) - \alpha_3 \beta_3 \tau_2 \right] \]  
(A32)

Moment equilibrium about the plastic hinge position requires
\[ M_p + N_{b1} e_1 - N_{b2} e_2 + N_{b2} e_3 = 0 \]  
(A33)

where \[ e_1 = Z_1 (1 - \frac{1}{2} \beta_1) \]  
(A34)

\[ e_2 = Z_1 + \frac{1}{2} \beta_2 Z_2 \]  
(A35)

\[ e_3 = Z_1 + \frac{1}{2} \beta_3 Z_2 + t_2 (1 - \frac{1}{2} \beta_3) \]  
(A36)

Finally, combining Eqs. A43 and A33 gives an expression for \( z_2 \):

\[ z_2 = t_2 \left( \frac{b + \sqrt{b^2 + ac}}{a} \right) \]  
(A37)

where

\[ a = (\alpha \beta) + \frac{\alpha_1}{\lambda} \left( \alpha_1 \beta_2^2 + \alpha_3 \beta_3^2 \right) \]  
(A38)

\[ b = \alpha_3 \beta_3 \left( \frac{\alpha}{\lambda} + \frac{\alpha}{\lambda} (\beta_3 - 1) \right) \]  
(A39)

\[ c = \frac{2M_p}{f_b d (\lambda t_2)^2} + \alpha_3 \beta_3 \left( \frac{2\alpha_1}{\lambda} \left( 1 - \frac{1}{2} \beta_3 \right) - \alpha_3 \beta_3 \right) \]  
(A40)

\[ \bar{a} \beta = \alpha_2 \beta_2 + \alpha_3 \beta_3 \]

The normal bearing load \( N_b \) for failure mode IV can be found in a similar manner to that described for failure mode III.
Fig. A1 Deformed shape and free body diagram of nail shank. Actual and equivalent bearing stresses on nail shank.
Fig. A2 Stress Block Factors $\alpha$ and $\beta$. 
Fig. A3 Failure Mode III.
Fig. A4 Flow Chart for solving normal bearing load equations.
Fig. A5 Failure Mode IV.
<table>
<thead>
<tr>
<th>Bearing Stress (MPa)</th>
<th>Fasting Emendment (mm)</th>
<th>BASIC DENs. (kg/m³)</th>
</tr>
</thead>
</table>
| 0.2 | 0.4 | 0.6 | 0.8 | 1.0 | 1.5 | 2.0 | 3.0 | 5.0 | 8 | 12 | 20 | 30 | 50 | 40 | 45 | 60 | 50 | 60 | 40 | 50 | 70 | 60 | 80 | 50 | 70 | 90 | 80 | 100 | 90 | 120 | 100 | 140 | 120 | **Bearing Stress Embden Characteristics**

1. 884mm Dia. Staple leg loaded in a direction parallel to the grain.
<table>
<thead>
<tr>
<th>Bearing stress (MPa)</th>
<th>FASTENER EMBEDDING (mm)</th>
<th>M/C</th>
<th>DENS.</th>
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</thead>
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<td>0.6</td>
<td>1.0</td>
</tr>
<tr>
<td>2.0</td>
<td>3.0</td>
<td>5.0</td>
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<th>FASTENER EMBEDDING (mm)</th>
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<th>DENS.</th>
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<td>1.0</td>
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<td>2.0</td>
<td>3.0</td>
<td>5.0</td>
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**Table continued...**
<table>
<thead>
<tr>
<th>PASTENER EMBEDMENT (mm)</th>
<th>M/C</th>
<th>BASIC</th>
<th>PERCENTAGE TO THE GRAIN</th>
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<tr>
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<tr>
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<td>0.4</td>
<td>0.6</td>
<td>1.0</td>
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<tr>
<td>FASTENER EMBRITTMENT (mm)</td>
<td>M/C DENS.</td>
<td>BASIC DENS.</td>
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Table continued....
### HEARING STRESS - EMBEDMENT CHARACTERISTICS

1.33 mm dia. nail loaded in a direction perpendicular to the grain

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<tr>
<th>MF</th>
<th>EMBEDMENT (mm)</th>
<th>M/C</th>
<th>DENS.</th>
<th>kg/m³</th>
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<td>1.0</td>
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<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
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| 18.9 | 23.9 | 27.1 | 28.5 | 30.9 | 34.8 | 38.7 | 46.2 | 53.2 | 59.2 |
| 21.6 | 24.1 | 26.1 | 30.0 | 30.0 | 35.0 | 38.7 | 45.7 | 52.9 | 59.0 |
| 18.5 | 18.9 | 20.1 | 21.0 | 21.8 | 24.4 | 26.1 | 31.2 | 35.8 | 41.1 |
| 16.2 | 24.2 | 30.9 | 34.6 | 38.9 | 54.4 | 62.1 | 74.2 | 80.2 | 84.6 |
| 23 | 24.3 | 33.8 | 37.7 | 41.1 | 48.3 | 53.9 | 63.6 | 68.4 | 71.1 |
| 23 | 24.3 | 33.7 | 34.7 | 41.1 | 45.9 | 50.5 | 56.8 | 64.3 | 71.1 |
| 18.9 | 24.2 | 27.6 | 29.7 | 31.4 | 36.0 | 43.6 | 44.7 | 48.8 | 51.7 |
| 17.4 | 25.4 | 27.1 | 28.5 | 28.8 | 29.0 | 30.5 | 41.1 | 52.0 | 56.1 |
| 16.2 | 23.0 | 27.1 | 29.0 | 31.2 | 38.4 | 44.2 | 53.7 | 63.1 | 71.4 |
| 16.9 | 22.5 | 24.7 | 26.6 | 28.0 | 31.7 | 36.0 | 45.7 | 53.2 | 59.0 |
| 14.7 | 22.0 | 28.5 | 33.4 | 39.6 | 50.3 | 58.5 | 69.8 | 77.6 | 81.0 |
| 16.2 | 20.8 | 22.0 | 23.9 | 26.3 | 29.0 | 34.6 | 38.7 | 40.8 | 42.0 |
| 20.4 | 26.3 | 29.2 | 31.7 | 33.8 | 39.2 | 44.0 | 51.7 | 55.6 | 58.2 |
| 18.9 | 25.6 | 30.2 | 32.4 | 33.6 | 38.7 | 40.0 | 52.4 | 59.9 | 64.8 |
| 24.2 | 32.6 | 41.1 | 47.1 | 54.4 | 64.3 | 74.0 | 84.8 | 90.4 | 92.8 |
| 20.5 | 31.9 | 36.7 | 40.6 | 45.9 | 50.8 | 61.6 | 69.1 | 73.1 | 74.3 |
| 18.9 | 26.6 | 37.7 | 43.0 | 43.3 | 48.6 | 52.7 | 59.0 | 63.1 | 66.2 |
| 17.4 | 24.7 | 30.5 | 35.8 | 39.2 | 41.8 | 46.4 | 54.4 | 60.9 | 66.7 |
| 21.5 | 27.8 | 31.2 | 33.6 | 36.0 | 40.6 | 45.4 | 53.9 | 59.9 | 62.8 |
| 25.2 | 28.6 | 30.1 | 33.4 | 35.5 | 39.9 | 44.0 | 52.1 | 57.5 | 63.6 |
| 15.2 | 21.0 | 27.8 | 30.5 | 31.4 | 34.6 | 39.2 | 47.1 | 52.4 | 55.1 |
| 19.8 | 30.2 | 38.9 | 41.6 | 46.2 | 52.2 | 57.8 | 63.8 | 67.7 | 68.4 |
| 18.9 | 25.1 | 31.9 | 34.6 | 36.0 | 40.8 | 44.2 | 49.5 | 53.7 | 56.8 |
| 23.7 | 28.5 | 31.6 | 36.1 | 46.4 | 52.2 | 59.9 | 66.2 | 72.0 | 74.0 |
| 16.9 | 24.7 | 29.0 | 32.1 | 34.8 | 40.1 | 44.0 | 50.7 | 60.2 | 62.4 |
| 18.4 | 25.1 | 29.7 | 34.1 | 37.2 | 44.5 | 50.0 | 58.0 | 62.1 | 64.5 |
| 16.0 | 24.2 | 28.5 | 31.2 | 34.6 | 44.4 | 50.0 | 56.4 | 60.2 | 64.0 |
| 14.5 | 19.8 | 23.0 | 25.9 | 26.3 | 31.7 | 33.8 | 40.8 | 48.3 | 55.6 |
| 14.3 | 25.1 | 31.4 | 35.8 | 38.4 | 44.5 | 49.8 | 56.3 | 60.9 | 64.5 |
| 18.9 | 27.8 | 30.9 | 34.6 | 36.0 | 40.3 | 43.3 | 49.8 | 59.9 | 67.7 |
| 19.6 | 27.1 | 31.9 | 35.8 | 38.7 | 46.2 | 52.4 | 57.8 | 62.8 | 68.6 |
| 18.4 | 25.9 | 31.9 | 34.8 | 37.5 | 43.3 | 47.1 | 58.2 | 66.7 | 70.8 |
| 23 | 28.3 | 40.4 | 44.7 | 48.3 | 57.0 | 65.7 | 79.5 | 85.6 | 88.5 |
| 21.0 | 28.3 | 42.4 | 44.2 | 50.0 | 68.2 | 72.0 | 74.0 | 76.0 | 78.0 |
| 14.5 | 23.9 | 27.8 | 30.5 | 34.3 | 39.6 | 45.0 | 52.9 | 59.5 | 65.5 |
| 16.4 | 22.2 | 25.1 | 27.3 | 30.0 | 34.1 | 38.9 | 45.4 | 51.1 | 54.1 |
| 12.6 | 17.6 | 20.1 | 22.2 | 23.0 | 25.6 | 29.5 | 32.6 | 35.0 | 37.9 |
| 23.0 | 31.6 | 39.0 | 45.9 | 50.3 | 58.2 | 65.5 | 73.2 | 78.8 | 83.4 |
| 23.7 | 32.1 | 36.7 | 39.6 | 42.8 | 49.1 | 56.6 | 67.2 | 76.1 | 81.0 |
| 17.2 | 29.0 | 33.8 | 37.7 | 42.5 | 49.3 | 58.0 | 65.0 | 71.5 | 75.9 |

Table continued...
APPENDIX C:

COMPUTER PROGRAM "EQUAKE"

A1 GENERAL

The EQUAKE program carries out step-by-step time-history analysis of a nonlinear single Degree of Freedom structure subjected to ground accelerations. It is coded in compatible Fortran.

Any consistent set of units may be used.

A2 INPUT FILE

1 Description of Analysis

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2 Description of Earthquake Record

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3 Hysteretic Control Parameters

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<td>Yield load $V_y$</td>
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<td>MUP</td>
<td>Ultimate load $V'$</td>
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<td>MI</td>
<td>Off-Set load $V_{OS}$</td>
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### Analysis Control Parameters

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<tr>
<td>NPRT</td>
<td>Output to the printer (or output file) is provided at the first time step and then every NPRT steps.</td>
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#### (c)

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### Hysteretic Model

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<td>1 = Elastoplastic oscillator</td>
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<td>2 = Pinching oscillator</td>
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### Output Control Parameters

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<td>WT</td>
<td>0 = Write time, restoring force and deflection to printer</td>
<td>I5</td>
</tr>
<tr>
<td></td>
<td>1 = Write time, base shear (restoring + damping force) and deflection to output file</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 = Write time, restoring force and deflection to output file</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3 = Write time, damping force and deflection to output file</td>
<td></td>
</tr>
</tbody>
</table>
### Restart

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Format</th>
</tr>
</thead>
<tbody>
<tr>
<td>RESTART</td>
<td>0 = Read control parameters for hysteretic model from restart file at the beginning of analysis</td>
<td>I5</td>
</tr>
<tr>
<td></td>
<td>1 = Read control parameters for hysteretic model from restart file at the beginning of the analysis, and write new control parameters for hysteretic model to restart file at the end of the analysis</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 = Write new control parameters for hysteretic model to restart file at the end of the analysis</td>
<td></td>
</tr>
</tbody>
</table>

### A3 SOURCE LISTING

A Fortran source listing of the computer program EQUAKE follows:
PROGRAM BY: W. SIEBERT
DATE: JULY 1984

***************************************************************

K0  INITIAL STIFFNESS
MP  YIELD FORCE +VE
FYN YIELD FORCE -VE
FP1 BI-LINEAR STIFFNESS +VE
FP2 BI-LINEAR FACTOR (2)
FP3 PINCHING STIFFNESS
FP4 UNLOADING FACTOR
BETTA SOFTENING FACTOR
RGAP INITIAL GAP +VE
RGAPM INITIAL GAP -VE
TTIME LENGTH OF E/Q RECORD
INTIM INVERSE OF TIME STEP
HPRT TIME STEPS BETWEEN OUTPUT
CRDAMP VISCOS DAMPING (%)
MMASS INTERIA MASS (KG)
INST INELASTIC BEHAVIOUR: 0 = ELASTIC
               1 = BI-LINEAR
               2 = PINCHING
TT(N) TIME
RT(N) DISPLACEMENT (M)
MT(N) FORCE (KN)
CT(N) VISCOS DAMPING FORCE

***************************************************************

INTEGER IEND, INDFL
REAL TITLE(l2), EQREC(12)
REAL MASS, PERIOD, NAFREQ, TTIME, CRDAMP
REAL VE, PS, PG
REAL TT(50000), RT(50000), MT(50000), CT(50000)
REAL CTL(50000), CT2(50000)
REAL XMAX, XMIN, FMAX, FIN
REAL T(100000), G(100000), GO, GGO
REAL KO, PPL, PP2, P3, PP4, BETTA, MYP, MYN, MUP
REAL M, MR, MI, MBP, MBN, MPG, MPN, MFF, MPN
REAL CRCHN, MCRCHN, RCRCHP, MRRCHP
REAL MRRVN, MRRVNP, MRRVCP, MRRCPN
REAL MRRVN, MRRVNP, MRRVCP, MRRCPN

READ IN DATA

READ (5, 600) TITLE
READ (5, 600) EQREC
READ (5, 610) KO
READ (5, 620) MYP, MUP, MI
READ (5, 620) PPL, PP2, P3, PP4, BETTA, RGAPP, RGAPM
READ (5, 630) TTIME, PG
READ (5, 650) INTIM, HPRT
READ (5, 650) CRDAMP
READ (5, 650) W
READ (5, 650) M
READ (5, 650) MTT
READ (5, 640) XMAX
READ (5, 650) XT
READ (5, 650) WTT
READ (5, 650) MT
READ (5, 650) MTT
READ (5, 650) MTT
READ (5, 650) RSTART

PRINT OUT DATA FOR CHECKING

IF(M, EQ, 1.0) GO TO 10
WRITE (6, 700) TITLE
WRITE (6, 710) EQREC
WRITE (6, 720) TTIME, INTIM, HPRT
WRITE (6, 725) PG
WRITE (6, 730) CRDAMP
WRITE (6, 735) C
WRITE (6, 740) IEND
MYP = MUP
IF (INSTR.EQ.0) WRITE (6, 745) KO
IF (INSTR.EQ.1) WRITE (6, 750) KO, MYP, MYN, PP1
IF (INSTR.EQ.2) WRITE (6, 760) KO, MYP, MUP, MI, PP1, PP2, P3, PP4,
                   BETTA, RGAPP, RGAPM
WRITE (6, 770) MASS

378
**** MAIN CONTROL PROGRAMME ****

DELTAT = 1.0 / INTIM
CRAEMP = 0.01 * Craemp
CALL DIGAC(T,G,INDD,PG)
NTMAX = (T'TIME * INTIM) + 1
NAFREQ = (100 * LOG2(MASS) / NAFREQ) ** 0.5
PERIOD = 5.238 / NAFREQ
IF(W.EQ.0) WRITE(6,799) PERIOD

C

IF(INYX,NE.0) GO TO 55
CALL INITIAL(F, J, VRO, ARO, R, M, RBP, RBN, MBP, MBN, RREC, P,
RRECV, RREC1, RRECH, MFZ, MFN, IFENDFL, GO, DRO)
1 DO 50 N = 1, NTMAX
IF(IFENDFL.EQ.1) GO TO 500
CALL GRIDAC(T, G, TIME, GDAR, IEND, DELTAT, GG, GO, N, J, IFENDFL)
CALL DYNAMC(DR, DRO, VRO, ARO, DELTAT, CRAEMP, NAFREQ, F,
GAR, M, RO, IFSIGN)
* CALL ELAST(N, R, F, DR, DRO, X0, RBP, RBN, MBP, MBN,
XIMAX, XIMIN, IFMAX, IFMIN, N, RT, MT)
* TT(N) = (N-1) * DELTAT
* CT(N) = CT(N) + (VRO * (2.0 * CRAEMP * MASS * NAFREQ)) / 1000.0
50 CONTINUE

C

55 IF(INYX,NE.1) GO TO 105
CALL INITIAL(F, J, VRO, ARO, R, M, RBP, RBN, MBP, MBN, RREC, P,
RRECV, RREC1, RRECH, IFENDFL, GO, DRO)
1 DO 100 N = 1, NTMAX
IF(IFENDFL.EQ.1) GO TO 500
CALL GRIDAC(T, G, TIME, GDAR, IEND, DELTAT, GG, GO, N, J,
IFENDFL)
CALL DYNAMC(DR, DRO, VRO, ARO, DELTAT, CRAEMP, NAFREQ, F,
GAR, M, RO, IFSIGN)
CALL BILIN(M, R, DR, DRO, MR, PK, IF1, IF2, MYP, MYN, RBP, RBN,
RREC, RRECV, XIMAX, XIMIN, IFMAX, IFMIN, N, RT, MT)
* TT(N) = (N-1) * DELTAT
* CT(N) = CT(N) + (VRO * (2.0 * CRAEMP * MASS * NAFREQ)) / 1000.0
100 CONTINUE

C

105 IF(INYX,NE.2) GO TO 500
CALL INITIAL(F, J, VRO, ARO, R, M, RBP, RBN, MBP, MBN, RREC, P,
RRECV, RREC1, RRECH, IFENDFL, GO, DRO)
1 IF(RSTART.LE.1) READ(4,770) PP3, PN3, PP5, PP11, PP12,
P, RBP
* IF(RSTART.LE.1) READ(4,770) RBN, MBP, MBN, RFN, MFZ, MFN,
RRECV
* IF(RSTART.LE.1) READ(4,770) RRECM, RREC1, RREC2, RREC3,
RRECV1, RRECV2, RREC3
* IF(RSTART.LE.1) READ(4,771) RRECV, RREC1, RRECM, RRECV,
100 CONTINUE
1 DO 200 N = 1, NTMAX
IF(IFENDFL.EQ.1) GO TO 500
CALL GRIDAC(T, G, TIME, GDAR, IEND, DELTAT, GG, GO, N, J,
IFENDFL)
CALL DYNAMC(DR, DRO, VRO, ARO, DELTAT, CRAEMP, NAFREQ, F,
GAR, M, RO, IFSIGN)
CALL BILIN(M, R, DR, DRO, MR, PK, IF1, IF2, MYP, MYN, RBP, RBN,
RREC, RRECV, XIMAX, XIMIN, IFMAX, IFMIN, N, RT, MT)
* TT(N) = (N-1) * DELTAT
* CT(N) = CT(N) + (VRO * (2.0 * CRAEMP * MASS * NAFREQ)) / 1000.0
200 CONTINUE

C

CONTINUE
IF(RSTART.GE.1) WRITE(3,770) PP3, PN3, PP5, PP11, PP12,
P, RBP
* IF(RSTART.GE.1) WRITE(3,770) RBN, MBP, MBN, RFN, MFZ, MFN,
RRECV
* IF(RSTART.GE.1) WRITE(3,771) RRECV, RREC1, RRECM, RREC2,
100 CONTINUE
* IF(RSTART.GE.1) WRITE(3,770) RRECV, RRECM, RREC1, RRECV
STOP

C

WRITE OUT RESULTS
IF(N.EQ.0) WRITE(6,820) XIMAX, XIMIN, IFMAX, IFMIN
IF(WT.EQ.0) WRITE(WT,820)
IF(WT.EQ.1) WRITE(WT,820)
IF(WT.EQ.2) WRITE(WT,820)
IF(WT.EQ.4) WRITE(WT,820)
STOP
FORMAT STATEMENTS

610 FORMAT (12A6)
610 FORMAT (F10.4)
620 FORMAT (7F10.4)
630 FORMAT (3F10.4)
640 FORMAT (15)
650 FORMAT (215)

700 FORMAT (12H1ITLE:, 10A6)
710 FORMAT (12H EO RECORD:, 10A6)
720 FORMAT (25H LENGTH OF E/O RECORD F8.2, ' SECONDS '/
* 25H INVERSE OF TIME STEP =, I8, ' PER SECOND '/
* 25H TIME STEPS BETWEEN O/P =, I8)
725 FORMAT (25H FG) ********** =, F10.4)
730 FORMAT (25H DAMPING (VISC) F8.2, ' PERCENT '
735 FORMAT (25H DAMPING FORCE (FRIC) =, F8.2, ' KN '
740 FORMAT (25H NUMBER OF CARDS  =, I8//)
745 FORMAT (21H0ELASTIC //
* 25H INITIAL STIFFNESS =, 1PE12.4/
* 25H INITIAL STIFFNESS =, 1PE12.4/
* 25H YIELD FORCE (+) =, 0PF12.3/
* 25H YIELD FORCE (-) =, F12.3/
* 25H BI-LINEAR FACTOR =, 0PF12.3/
760 FORMAT (25H0PICHING WITH ELASTICITY //
* 25H INITIAL STIFFNESS =, 1PE12.4/
* 25H YIELD LOAD =, 0PF12.3/
* 25H ULTIMATE LOAD =, F12.3/
* 25H PINCHING LOAD =, F12.4/
* 25H BI-LINEAR FACTOR =, 0PF12.3/
* 25H TRI-LINEAR FACTOR =, F12.3/
* 25H PINCH FACTOR =, F12.3/
* 25H UNLOADING FACTOR =, F12.3/
* 25H STIF. DEGRADATION FACT =, F12.3/
* 25H INITIAL GAP =, F12.3/
* 25H INITIAL GAP =, F12.3/
770 FORMAT (8P10.4)
771 FORMAT (7P10.4, 110)
790 FORMAT (25H MALL) *********** =, 1PE12.4/
799 FORMAT (35H PERIOD) ********** =, 0P7.4//
800 FORMAT (1HL 'TIME (SEC)' , 6X, 'DISPLACEMENG(M)' , 2X, 'FORCE (KN)' )'
820 FORMAT (36H MAXIMUM VALUES OBTAINED IN ANALYSIS//
* 30H DISPLACEMENT +VE =, 0PF8.3/
* 30H DISPLACEMENT -VE =, F8.1/
* 30H FORCE +VE =, 0PF8.3/
* 30H FORCE -VE =, F8.3/

CLOSE (2, DISP = CRUNCH)
CLOSE (3, DISP = CRUNCH)
END
SUBROUTINE DIGAC (T,G,IEND,FG)

******************************************************************************
SUBROUTINE TO READ DIGITIZED ACCELERATION RECORDS
******************************************************************************

REAL T(100000),G(100000),STAR(12)

T   TIME
G   GROUND ACC (IN TERMS OF GRAVITY)
ICC  CARD NUMBER
J    RECORD NUMBER

IEND = 0
READ(1,610) E0G
DO 150 J=1,1000
   READ(1,600,E0G) ISEQ,(T(J),G(J),J=4*I-3,4*I)
   IF(SEQ.EQ.ICC) GO TO 150
   WRITE(6,700) ISEQ,IICC
150 CONTINUE
IEND = I-1
600 FORMAT(I3,4(F8.4,P9.6))
DO 151 LOOP = 1,100000
151 G(LOOP) = G(LOOP) * FG
610 FORMAT(12A6)
700 FORMAT(120,22HDIGITIZED E/Q CARD NO.,I3,'IS WHERE CARD NO.=',I3,11SHOULD BE')
RETURN
END
SUBROUTINE GRDACC (T,G,TIME,GOAR,IEND,DELTAT,GG,GGO,N,J,IENDPL)

*******************************************************************
SUBPROGRAMME TO CALCULATE GROUND ACCELERATIONS
*******************************************************************

REAL T(100000),G(100000),TIME,GOAR,DELTAT,DELTI

TIME = (N-1)*DELTAT
IF(TIME.LT.T(J+1)) GO TO 50

40 J = J+1
50 IF(J.GT.(4*IEND)) GO TO 150

TS = T(J+1)-T(J)
GS = G(J+1)-G(J)
IF(TS.LE.0.0) GO TO 40
DELTI = TIME-T(J)
GG = G(J)+(GS*DELTI/TS)
GO TO 300

150 GG = 0.0

300 GOAR = (GG-GGO)*9.81
GGO = GG
RETURN
C
END
SUBROUTINE INITIAL (F, J, VRO, ARO, R, M, RBP, RBN, MBP, MBN, RRECIP, RRECNI, MFP, MFN, IENDFL, GGO, DRO)

* SUBPROGRAMME TO INITIALISE VARIABLES *
* ******************************************

F = 1.0
J = 1
VRO = 0.0
ARO = 0.0
R = 0.0
M = 0.0
RBP = 0.0
RBN = 0.0
MBP = 0.0
MBN = 0.0
RRECIP = 0.0
RRECNI = 0.0
MFP = 0.0
MFN = 0.0
IENDFL = 0.0
GGO = 0.0

RETURN

END
SUBROUTINE DYNAMIC (DR, DRO, VRO, ARO, DELTAT, CRDAMP, NAFREQ, F, GOAR, MR, KO, ISIGN)

**********************************************************************************************
SOLUTION TO DYNAMIC EQUATION OF MOTION
**********************************************************************************************

REAL EQSTIP, EQLD, DR, DRO, VRO, ARO, DELTAT, CRDAMP, NAFREQ, F, MR

ARO INITIAL ACCELERATION
VRO INITIAL VELOCITY
MR FORCE OVER-RUN
DR INCR DISPLACEMENT
GOAR GROUND ACCELERATION

CALCULATE INCREMENT IN DISPLACEMENT USING NEWMARK INTEGRATION SCHEME

100 EQST1 = 4.0/(DELTAT*DELTAT)
EQST2 = (4.0*CRDAMP*NAFREQ)/DELTAT
EQST3 = NAFREQ*NAFREQ*F
EQSTIP = EQST1*EQST2*EQST3
EQLD1 = (4.0*VRO)/DELTAT
EQLD2 = 2.0*ARO
EQLD3 = 4.0*CRDAMP*NAFREQ*VRO
EQLD4 = MR*NAFREQ*NAFREQ/KO
EQLD = EQLD1+EQLD2+EQLD3+EQLD4-GOAR
DR = EQLD/EQSTIP

CALCULATE NEW INITIAL CONDITIONS

250 VRO1 = (2.0*DR/DELTAT)-VRO
ARO1 = (4.0*DR/(DELTAT*DELTAT))-(4.0*VRO/DELTAT)-ARO
VRO = VRO1
ARO = ARO1
IF (K.GE.2) WRITE (6, 965)
GO TO 300

200 IF (K.GE.2) GO TO 250
ISIGN = ISIGN
K = K+1
GO TO 100

300 RETURN

963 FORMAT (40H SOLUTION OF EQUATION OF MOTION UNSTABLE)

END
SUBROUTINE ELASTI(N, R, F, DR, DRO, KO, RBP, RBN, MBP, MBN,
    XIMAX, XMIN, FIMAX, FMIN, N, RT, MT)

*****************************************************************************
ELASTIC STIFFNESS MODEL
*****************************************************************************

REAL F, RT(5000), MT(5000), R, M, DR, DM, RBP, RBN, MBP, MBN

F = 1.0
DM = DR*KO
M = M+DM
R = R+DR
IF (R.GT.RBP) RBP = R
IF (R.LT.RBN) RBN = R
RT(N) = R

IF (M.GT.MBP) MBP = M
IF (M.LT.MBN) MBN = M
MT(N) = M
IF (RT(N).GT.XIMAX) XIMAX = RT(N)
IF (RT(N).LT.XMIN) XMIN = RT(N)
IF (MT(N).GT.FIMAX) FIMAX = MT(N)
IF (MT(N).LT.FMIN) FMIN = MT(N)

RETURN

END
SUBROUTINE BILINR (M, R, DR, DRO, NR, F, KO, P, MYP, MYN, RBP, RBN, RRECP, RECPN, XIAX, XIAM, PLMA, P, IMA, IMA, R, MT)  

REAL RT (50000), RT (50000), R, M, DR, DM, RBP, RBN, MYP, MYN, KO, P

REAL MYP, MYN

RYP = MYP / KO
RYN = MYN / KO
RR = RYP - RYN
MR = 0.0
DM = KO * DR

ELASTIC CASE

IF (RBP > RT, RYP, OR. RBN LT. RYN) GO TO 50
IF (R(DR, LT. RYP)) GO TO 10
IF (R(DR, LT. RYN)) GO TO 20
M = M + DM
F = 1.0
GO TO 1000

EXCEEDING +VE YIELD FIRST TIME

10 DRP = R + DR - RYP
M = MYP * DRP * KO
GO TO 21

EXCEEDING -VE YIELD FIRST TIME

20 DRP = R + DR - RYN
M = MYN - DRP * KO
GO TO 1000

LOADING PLASTICALLY

50 IF (F NE. P) GO TO 100
IF (DR > DRO, LT. 0.0) GO TO 60
M = M + P * KO + DR
F = P
GO TO 1000

UNLOADING FROM MONOTONIC CURVE

60 IF (DR GT. 0.0) GO TO 70
RRECP = R
RECPN = RRECP - RR
IF (ABS (DR) .GE. ABS (RR)) GO TO 64
62 M = M + DM
MR = DM * (F - 1.0)
GO TO 1000

64 RA = DR - RR
RM = P * KO * RA
RMR = P * DM * MYP - MYN + RM
M = M - MYP + MYN + RM
F = P
GO TO 1000

70 RRECN = R
RRECP = RRECN + RR
IF (ABS (DR) .LT. ABS (RR)) GO TO 62
RA = DR - RR
RM = P * KO * RA
RMR = P * DM * MYP - MYN + RM
M = M - MYP + MYN + RM
F = P
GO TO 1000

ON UNLOADING PATH

100 IF (DR > DRO) 150, 1000, 110
110 IF (DR LT. 0.0) GO TO 130
111 IF (R + DR GE. RREV) GO TO 120
115 M = M + DM
F = 1.0
GO TO 1000

120 RR = R + DR - RREV
RM = KO * (DR - RR + RR * P)
M = M + RM
MR = DM - RR
F = P
GO TO 1000

C 130 RREV = RRECP - RR
131 IF (R + DR LE. RREV) GO TO 120
GO TO 115

GO TO 1000
C 150 IF(DR.GT.0.0) RREV = RREC P
    IF(DR.LT.0.0) RREV = RREC N
    IF(DR.EQ.0.0) RREV = RREV
    IF(DR.GT.0.0) GO TO 111
    IF(DR.LT.0.0) GO TO 131
CLEAN-UP
C
1000 R = R + DR
    IF(DR.NE.0.0) DRO = DR
    IF(R.GT.RBP) RBP = R
    IF(R.LT.RBN) RBN = R
    IF(M.GT.MBP) MBP = M
    IF(M.LT.MBN) MBN = M
    RT(N) = R
    MT(N) = M
    IF(RT(N).GT.X1MAX) X1MAX = RT(N)
    IF(RT(N).LT.X1MIN) X1MIN = RT(N)
    IF(MT(N).GT.F1MAX) F1MAX = MT(N)
    IF(MT(N).LT.F1MIN) F1MIN = MT(N)
C
RETURN
C
END
SUBROUTINE INSLAC(N, R, DR, DRO, KO, P3, PPL, PP2, PN3, PPI, PP4, PP5, PPL1, 00045500 1 PPPL, BETTA, MYP, MYP, MI, MP, P, ABP, RBN, MBP, HBN, 00045460 2 RGAPP, RGANN, RFP, RFN, MFP, MFN, RREC, RRCNP, RRCNP, RRCNP, RRCNP, RRCNP, RRCNP, RRCNP, 00045470 3 RREEV, RREEV, RREEV, RREEV, RREEV, RREEV, RREEV, RREEV, RREEV, RREEV, 00045500 4 XMAX, XMIN, PMAX, PMIN) 00046030 5 XMAX, XMIN, PMAX, PMIN) 00046030 6 *** STIFFNESS MODEL - PINCHING WITH INCREASING SLACKNESS 00045650 7 *** 00046200 8 PROGRAMMED BY : WAYNE STEWART 00046300 9 DATE/VERSION : APRIL 1984/L 00046400 10 00046500 11 REAL F, R, N, DR, DM, ABP, RBN, MBP, HBN, MI, KO, MR 00046800 12 REAL RFP, RFN, MFP, MFN 00046900 13 REAL RGAPP, RGANN, RFP, RFN, MFP, MFN, RREC, RRCNP, RRCNP 00047100 14 REAL RREEV, RREEV, RREEV, RREEV, RREEV, RREEV, RREEV, RREEV, RREEV, RREEV 00047200 15 INTEGER NFLAG, RT, MT, 00047300 16 INTEGER NFLAG, RT, MT, 00047300 17 INITIAL COMMON CALCULATIONS 00047400 18 RYP = (MYP/KO)*RGAPP 00047500 19 RYN = (-MYP/KO)*RGAPN 00047600 20 IF (PPL.EQ.0.0) GO TO 1 00047900 21 HUP = RYP+(MUP-MYP)/(PPL*KO) 00047910 22 RUN = RYN-(MUP-MYP)/(PPL*KO) 00048090 23 RUN = RYN-(MUP-MYP)/(PPL*KO) 00048090 24 GO TO 2 00048100 25 1 RUP = RYP 00048100 26 RYN = RYN 00048100 27 2 IF (PPL.EQ.1.0) WRITE(6,1910) 00048200 28 IF (PPL.EQ.1.0) WRITE(6,1910) 00048200 29 IF (PPL.EQ.1.0) WRITE(6,1910) 00048200 30 IF (PPL.EQ.1.0) WRITE(6,1910) 00048200 31 PMAX = 10 00048300 32 PMAX = 10 00048300 33 DM = KO*F*DR 00048700 34 QM = R*DM 00048800 35 1 ELASTIC 00048900 36 IF (FLAG.NE.0.0) GO TO 75 00049000 37 IF (ABP.GT.0.0) GO TO 30 00049300 38 IF (R+DR.GT.RYP) GO TO 10 00049400 39 IF (R+DR.GT.RYN) GO TO 20 00049500 40 PM = 0 00049600 41 IF (R+DR.GT.RGAPP) PM = KO*(R+DR-RGAPP) 00049700 42 IF (R+DR.GT.RGAPP) PM = KO*(R+DR-RGAPP) 00049700 43 IF (R+DR.GT.RGAPP) PM = KO*(R+DR-RGAPP) 00049700 44 F = 0.0 00049800 45 IF (R+DR.GT.RGAPP) F = 1.0 00049900 46 GO TO 1000 00050000 47 1A EXCEEDING POSITIVE ELASTIC DISP - FIRST TIME 00050100 48 1B EXCEEDING NEGATIVE ELASTIC DISP - FIRST TIME 00050200 49 20 IF (R+DR.GT.RY) GO TO 25 00051200 50 PR = R+DR-RY 00051300 51 PM = -MYP+PR*KO*PP1 00051400 52 F = PP1 00051500 53 FLAG = 1 00051600 54 GO TO 1000 00051700 55 1A EXCEEDING POSITIVE YIELD - FIRST TIME 00052100 56 1B EXCEEDING NEGATIVE YIELD - FIRST TIME 00052200 57 25 PR = R+DR-RUN 00052900 58 PM = -MUP+PR*KO*PP2 00053000 59 F = PP2 00053100 60 FLAG = 2 00053200 61 GO TO 1000 00053300 62 1E ELASTIC IN ONE DIRECTION ONLY 00053400 63 30 IF (R.GE.0.0) GO TO 40 00053900 64 IF (R+DR.GE.RYP) GO TO 10 00054000 65 IF (R+DR.GE.RREEV) GO TO 255 00054100 66 GO TO 5 00054200 67 40 IF (R.GE.0.0) GO TO 40 00054300 68 IF (R+DR.GE.RREEV) GO TO 280 00054400 69 GO TO 5 00054500
75 GO TO (85, 95, 100, 320, 1000, 200, 215, 220, 225, 230, 275, 250) FLAG

85 IF(R+DR. GE. RUP) GO TO 15
IF(R+DR. LE. RUMP) GO TO 25
IF(DR+DRO. LT. 0.0) GO TO 100
PM = M+DR*KO*PPL
F = PPL
GO TO 1000

95 IF(DR+DRO. LT. 0.0) GO TO 100
PM = M+DR*KO*PP2
F = PP2
GO TO 1000

100 IF(DR+GT. 0.0) GO TO 150
MBP = M
RPP = RGAPP+((R-RGAPP)*BETTA)
MPP = ((RPP-RYP)*KO*PPL)+MYP
IF(RPP+GT. RUP) MPP = ((RPP-RUP)*KO*PP2)+MYP

105 RRECP = RBP-MBP/(KO*PP4)

107 IF(RBN. GE. RYN) GO TO 105
PN3 = (RYN/RPN)**P3
RRECN1 = MPN+MI-RPN*KO*PN3
RRECN2 = M1/RRECP-KO*PN3
GO TO 106

109 IF(3R+GE. 2.0*MI) AND (3R+LE. 0.0) GO TO 109
IF(R+GT. RYN) GO TO 107
IF(RGAN. GE. 2.0) GO TO 108
MRECN = -MYP
RRECN = RYN
PPL1 = (MRECN/(RRECN-RRECP))/KO
GO TO 109

108 RRECN = RAGAN
MRECN = 0.0
PPL1 = 0.0

110 IF(DR+LT. RRECP) GO TO 110
PM = MBP + KO*DR*PP4
F = PP4
FLAG=6
GO TO 1000

115 IF(DR+HT. RRECN) GO TO 115
PM = R+DR-RRECP
F = PK*KO*PPL
F = PPL1
FLAG = 11
GO TO 1000

120 IF(RBN. LT. RYN) GO TO 140
IF(R+DR. LT. RYN) GO TO 130
PM = R+DR-RGAPN
PM = PK*KO
F = 1.0
FLAG = 0
GO TO 1000
390

130 IF(R+DR.LE.RUN) GO TO 135
PR = R+DR-RYN
PM = -MYP+PR*KO*PP1
F = PP1
FLAG = 1
GO TO 1000

135 PR = R+DR-RUN
PM = -MUP+PR*KO*PP2
F = PP2
FLAG = 2
GO TO 1000

3E UNLOAD TO PINCH
140 IF(R+DR.LE.RPH) GO TO 145
PR = R+DR-RRECN
PM = RRECH+PR*KO*PN3
F = PN3
FLAG = 4
GO TO 1000

3F UNLOAD TO MONOTONIC VIA PINCH
145 IF(R+DR.LE.RUN) GO TO 147
PR = R+DR-RYN
PM = -MYP+PR*KO*PP1
F = PP1
FLAG = 1
GO TO 1000

147 PR = R+DR-RUN
PM = -MUP+PR*KO*PP2
F = PP2
FLAG = 2
GO TO 1000

4 UNLOADING FROM -VE MONOTONIC CURVE
150 MBN = M
RPN = RGAPH+((R-RGAPN)*BETTA)
MPN = (RPN-RYN)*KO*PP1-MYP
IF(RPN.LT.RUN) MPN = ((RPN-RUN)*KO*PP2)-MUP

RRECN1 = MBN-MBN/(KO*PP4)
IF(RBP.LE.RYN) GO TO 151
PP3 = (RYP/RYP)*PP3
RRECP1 = MPP-MI-RPP*KO*PP3
RRECP2 = -M/MRRECN=KO*PP3
GO TO 152

151 RRECP1 = -M-RGAPP*EQ
RRECP2 = -M/MRRECN-KO
152 RRECP = RRECP1/RRECP2
PP12 = -(MI/MRRECN)/KO
MRECP = PP12*KO*RRECP-MI

INTERSECTION POINT EXCEEDS 2*MI
IF((RRECP.LE.2.0*MI) OR (RRECP.GE.0.0)) GO TO 154
IF(RBP.GT.RYN) GO TO 153
MRRECP = HYP
RRECP = HYP
PP12 = (RRECP/(RRECP-RRECN))/KO
GO TO 154

153 MRRECP = 2.0*MI
RRECP1 = RPP*KO*PP3-MPP+2.0*MI
RRECP2 = KO*PP3
RRECP = RRECP1/RRECP2
PP12 = (MRRECP/(RRECP-RRECN))/KO
GO TO 154

154 MRRECP = RGAPP
MRECN = 0.0
PP12 = 0.0

4A UNLOAD TO UNLOADING CURVE
155 IF(R+DR.GT.RRECN) GO TO 155
PM = MBN+KO*DR*PP4
F = PP4
FLAG=6
GO TO 1000

4B UNLOAD TO SLIP
154 IF(R+DR.GT.RRECP) GO TO 160
PR = R+DR-RRECN
PM = PR*KO*PP12
F = PP12
FLAG = 12
GO TO 1000

00063130 00063200
00063300 00063400
00063550 00063600
00063700 00063800
00063900 00064000
00064100 00064200
00064300 00064400
00064500 00064600
00064700 00064800
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00069700 00069800
00069900 00070000
00070100 00070200
00070300 00070400
00070500 00070600
00070700 00070800
00070900 00071000
00071100 00071200
00071300 00071400
00071500 00071600
00071700
4C UNLOAD TO ELASTIC

160 IF (RRP GT RYP) GO TO 180
IF (R+DR GT RYP) GO TO 170
PR = R+DR-RYP
PM = R*KO
F = 1.0
FLAG = 0
GO TO 1000

4D UNLOAD TO MONOTONIC VIA ELASTIC

170 IF (R+DR GE RUP) GO TO 175
PR = R+DR-RYP
PM = R*PR*R+KO*PP1
F = PP1
FLAG = 1
GO TO 1000

175 PR = R+DR-RUP
PM = R*PR*R+KO*PP2
F = PP2
FLAG = 2
GO TO 1000

4E UNLOAD TO PINCH

180 IF (R+DR GE RPP) GO TO 190
PR = R+DR-RPP
PM = R*PR*R-RECUP
F = PP2
FLAG = 3
GO TO 1000

4F UNLOAD TO MONOTONIC VIA PINCH

190 IF (R+DR GE RUP) GO TO 195
PR = R+DR-RYP
PM = R*PP*PR*R+KO*PP1
F = PP1
FLAG = 1
GO TO 1000

195 PR = R+DR-RUP
PM = R*PP*PR*R+KO*PP2
F = PP2
FLAG = 2
GO TO 1000

5 UNLOADING CURVE

5A POSITIVE UNLOADING CURVE - FLAG=6

200 IF (R.LB .0 .0 ) GO TO 210
IF (R+DR LT RRECUP) GO TO 110
IF (R+DR LT RBP) GO TO 190
PM = R+KO*DR*PP4
F = PP4
GO TO 1000

5B NEGATIVE UNLOADING CURVE - FLAG=6

210 IF (R+DR LT RRECNU) GO TO 155
IF (R+DR LT RBN) GO TO 145
GO TO 205

5C POSITIVE UNLOADING FROM PINCH - FLAG=7

215 IF (R+DR LT RREWH) GO TO 241
IF (R+DR, LE RFP) GO TO 180
GO TO 205

5D NEGATIVE UNLOADING FROM PINCH - FLAG=8

220 IF (R+DR LT RREW) GO TO 245
IF (R+DR LE RFP) GO TO 140
GO TO 205

5E POSITIVE UNLOADING FROM SLIP - FLAG=9

225 IF (R+DR LT RREW) GO TO 241
IF (R+DR LT RFP) GO TO 235
GO TO 205

5F NEGATIVE UNLOADING FROM SLIP - FLAG=10

230 IF (R+DR LT RREW) GO TO 245
IF (R+DR LT RFP) GO TO 240
GO TO 205

5G LOADING ONTO (+VE) SLIP FROM UNLOADING CURVE

235 IF (R+DR GE RRECP) GO TO 160
PM = M*PR+R+KO*PP12
F = PP12
FLAG = 12
GO TO 1000
5H LOADING ONTO (-VE) SLIP FROM UNLOADING CURVE

240 IF((R+DR.RLE.RRECNI) GO TO 120
PM = MNP+DR*K0*PP11
F = PP11
FLAG = 11
GO TO 1000

241 IF((R+DR.RLE.RRECNI) GO TO 120
PR = R+DR.RREWN
PM = RREWN+PR*K0*PP11
F = PP11
FLAG = 11
GO TO 1000

245 IF((R+DR.RLE.RRECNI) GO TO 120
PR = R+DR.RREWP
PM = RREWP+PM*K0*PP12
F = PP12
FLAG = 12
GO TO 1000

6A SLIP - DISPLACEMENT IN POSITIVE DIRECTION - FLAG = 12

250 IF(DR+DRO.LT.0.0) GO TO 255
251 IF((R+DR.RLE.RRECNI) GO TO 160
PM = M+DR*K0*PP11
F = PP12
GO TO 1000

6B CHANGE IN LOADING DIRECTION (+VE CURVE)

255 MFP = M
MFP = R
IF (RRECU.OP.O.0.0) GO TO 262
RREWN1 = M+MFP-K0*PP4+RFP
RREWN2 = M/RRECU-K0*PP4
RREWN = RREWN1/RREWN2
RREWN = RREWN*PP4-RFP*PP4*PP4+MFP
IF (RWN.GE.RWN) GO TO 256
RNP = (RWN/RPN)+*P3
RRECN1 = MP+M-RNP*PP4*K0*PN3
RRECN2 = M1/RRECU-KO*PN3
GO TO 257
256 RRECN1 = M+RGNP*KO
RRECN2 = M/RRECU-KO
RRECN = RRECN1/RRECN2
PP11 = (M1/RRECU)/KO
MREP = PP11*K0*RRECN-M1
GO TO 258

INTERSECTION POINT EXCEEDS 2*M

258 RRECN = -2.0*M
RRECN1 = RNP*K0*PP3+MPN-2.0*M
RRECN2 = K0*PN3
RRECN = RRECN1/RRECN2
GO TO 259
259 RRECN = RGNP
RRECN = 0.0
259 RRECN = (M-RRECN)/RRECU/PP4
RREWN1 = RRECU+K0*PP11+MFP-K0+PP4*RFP
RREWN2 = K0*PP11-PP4
RREWN = RREWN1/RREWN2
RREWN = RREWN*PP4-RFP*PP4+MFP
GO TO 258
260 IF(RREWN.LE.RRECNI) GO TO 270
261 IF((R+DR.RLE.RREWNI) GO TO 241
PM = MFP+DR*K0*PP4
F = PP4
FLAG=9
GO TO 1000

LOAD TO SLIP ALTHOUGH NO YIELDING IN +VE DIRECTION

262 RREWN1 = M+MFP-K0*PP4*RFP
RREWN2 = K0*PP4
RREWN = RREWN1/RREWN2
RREWN = -M
RNP = (RWN/RPN)+*P3
RRECN1 = RNP*K0*PP3-MPN-M1
RRECN2 = K0*PN3
RRECN = RRECN1/RRECN2
RRECN = M1
GO TO 260
6C LOAD ONTO PINCH -VE

270 RREWNI = HFP*MHN*KO*(RPN*PN3-RFP*PP4)
RREWNI = KO*(PP1-PP4)
RREW = RREWNI/RREW2
RRECNI = RREW
RRECCH = KO*(RREW*PP4-RFP*PP4)+MPP
GO TO 261

6D SLIP - DISPLACEMENT IN NEGATIVE DIRECTION - FLAG = 11

275 IF (DR+DRO.LT.0.0) GO TO 280
IF (R+DR.LT.RRECNI) GO TO 120
PM = M+DR*KO*PP11
F = PP11
GO TO 1000

6E CHANGE IN LOADING DIRECTION (-VE)

280 MFP = M
RPN = R
IF (RRECNU.EQ.0.0) GO TO 287
RRECNI = -MI+MFP-KO*PP4*RFN
RRECNI = -MI+RRECNU-KO*PP4
RREP = RREP1/RREP2
RREP = RREP*PP4*RFN*KO*PP4+MFP
IF (RFN.LT.RFP) GO TO 28L
PP3 = (RFP/RFN)*PP3
RREP = RREP*PP4*RFN*KO*PP4
RREP = -MI+RRECNU-KO*PP3
RREP = RREP1/RREP2
GO TO 282

281 RREP1 = -MI-RRAPP*KO
RREP2 = -MI+RRECNU-KO

282 RRECP = RRECP1/RRECP2
PP12 = -(MI/RRECNU)/KO
MARECP = PP12*KO*RRECP+MI

INTERSECTION POINT EXCEEDS 2*MI

283 RRECP = 2.0*MI
RRECP = RRECP1=PP1+MPP+2.0*MI
RRECP = KO*PP3
RRECP = RRECP1/RRECP2
GO TO 284

284 RRECP = RRECP1/RRECP2
RARE = 0.0

285 IF (RREWP.GE.RRECP) GO TO 290
IF (R+DR.GE.RREWP) GO TO 245
PM = MFN+DR*KO*PP4
F = PP4
FLAG = 10
GO TO 1000

LOAD TO SLIP ALTHOUGH NO YIELDING IN -VE DIRECTION

287 RREP1 = -MI+MFP-RFN*KO*PP4
RREP2 = -KO*PP4
RREP = RREP1/RREP2
MAREP = MI
PP3 = (RFP/RFN)*PP3
RRECP = RRECP1=RRECP3+MPP+MI
RRECP = KO*PP3
RARECP = RARECP1/RARECP2
MAREP = MI
GO TO 285

6F LOAD ONTO PINCH +VE

290 RREP1 = MFP-HPP*KO*(RFP*PP3-RFN*PP4)
RREP2 = KO*(PP1-PP4)
RREP = RREP1/RREP2
MAREP = KO*(RREP*PP4-RFP*PP4)+MFP
GO TO 286
7 PINCHING - FLAG = 3
7A LOADING +VE
300 IF (DR*DROP.LT.0.0) GO TO 360
IF (R+DR.GE.RPP) GO TO 190
PM = M+DR*KO*PP3
P = PP3
GO TO 1000
7B LOADING -VE
320 IF (DR*DROP.LT.0.0) GO TO 350
IF (R+DR.LE.RPN) GO TO 145
PM = M+DR*KO*P3
P = PM3
GO TO 1000
7C UNLOADING -VE

350 MNF = M
RFN = R
RREP1 = M+RFN-KO*PP4*RFN
RREP2 = M+RFN-KO*PP4*RFN
351 RREP = RREP1/RREP2
MRREP = RREP*KO*PP4-RPN*KO*PP4+RFN
IF (RPN.GE.RNP) GO TO 352
PP1 = (RNP/RPN)**PP3
MRRCP1 = MPP-MI-RPP*KO*PP3
MRRCP2 = M+MREP-KO*PP3
GO TO 353
352 MRRCP1 = MI-RGAPP*KO
MRRCP2 = M+MREP-KO*PP3
353 MRRCP = MRRCP1/MRRCP2
PP12 = (MI/MRRCP)/KO
MRRCP = PPL*KO*MRRCP*MI

INTERSECTION POINT EXCEEDS 2*MI

356 IF (RREP.GE.RREP) GO TO 245
PM = NPP*DR*KO*PP4
F = PP4
FLAG=8
GO TO 1000

7D UNLOADING +VE

360 MPP = M
RFN = R
RREP1 = M+RFN-KO*PP4*RFN
RREP2 = M+RFN-KO*PP4*RFN
361 RREP = RREP1/RREP2
MRREP = RREP*KO*PP4-RPN*KO*PP4+MPP
IF (RPN.GE.RYN) GO TO 362
PN3 = (RPN/RYN)**PP3
MRRCP1 = MPP-MI-RPN*KO*PN3
MRRCP2 = MI/MRRCPU-KO*PN3
GO TO 363
362 MRRCP1 = MI-GRAPP-KO
MRRCP2 = MI/MRRCPU-KO
363 MRRCP = MRRCP1/MRRCP2
PP11 = (MI/MRRCPU)/KO
MRRCP = PPL*KO*MRRCP-MI

394
INTERSECTION POINT EXCEEDS 2*MI

IF(MRECN.GE.-2.0*MI.AND.RRECN.LE.0.0) GO TO 366
IF(RBPN.LT.RTH) GO TO 364
IF(RGAPN.NE.0.0) GO TO 3641

RRECN = -MYP
RRECN = RNW
GO TO 365

364 RRECN = -2.0*MI
RRECN1 = RPN*KO+PN3-MPN-2.0*MI
RRECN2 = KO*PN3
RRECN = RRECN1/RRECN2
GO TO 365

3641 RRECN = RGAPN
MRECN = 0.0

365 PP11 = (MRECN/(RRECN-RRECPU))/KO
RREWN1 = RRECPU*KO*PP11-MPP-KO*PP4*AFP
RREWN2 = KO*(PP11-PP4)
RREWN = RREWN1/RREWN2
RREWN = RREWN*KO*PP4-MPP*KO*PP4+MPP

366 IF(RL.NE.RREWN) GO TO 241
FN = MFP+DR*KO*PP4
F = PP4
FLAG=7
GO TO 1000

CLEAN UP

1000 R = R+DR
M = PM
MR = QM-PM
IF(DR.NE.0.0) DRO = DR
IF(R.GT.RBP) RBP = R
IF(R.LT.RBN) RBN = R
RT(N) = R
MT(N) = N
IF(RT(N).GT.XI(MAX)) XI(MAX)=RT(N)
IF(RT(N).LT.XI(MIN)) XI(MIN)=RT(N)
IF(MT(N).LT.FIN(MIN)) FIN(MIN)=MT(N)

WRITE (6,690) L,FLAG,MR,DR,MR,F,RRG,RRP,RRP,RR,YR,RR,YR,RR
690 FORMAT(LH.2I3,F8.3,F2P7.3,F8.3,F5.2,12P7.3)

CONTINUE

1910 FORMAT("**********ERROR PP1 & PP2 CAN NOT BE EQUAL TO 1.0**********")
1920 FORMAT("**********ERROR PP1 & PP2 CAN NOT BE EQUAL TO 1.0**********")
1930 FORMAT("**********ERROR PP3 CAN NOT BE LESS THAN 1.0 **********")
1940 FORMAT("**********ERROR PP4 CAN NOT BE LESS THAN 1.0 **********")

RETURN
END