Displacement Behaviour of Reinforced Concrete Walls in Regions of Lower Seismicity

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Abstract

Displacement-based seismic design offers many benefits over traditional force-based procedures in lower seismic regions. This is largely due to the lower displacement demands that result from the upper magnitude limit of intraplate earthquakes. Displacement-based design approaches require a comprehensive understanding of the displacement behaviour of structures, which has historically been largely under-researched in regions of lower seismicity.

Reinforced concrete (RC) construction is the dominant form of construction for multi-residential and commercial buildings in Australia, where the majority of low, mid and high-rise buildings rely on RC walls as the primary lateral load resisting system. The walls are typically designed for strength, without consideration for nonlinear behaviour, resulting in lateral load resisting systems with limited ductility. The primary objectives of this research project are to develop a better understanding of the displacement behaviour of limited ductile RC walls and to develop simplified expressions for calculating the force-displacement response of walls. Furthermore, this thesis aims to generally improve the seismic design and detailing of RC wall buildings in Australia.

The displacement behaviour of limited ductile RC walls has been assessed firstly, using a comprehensive experimental testing program and secondly, using analytical models, which have been developed and verified against the experimental results. The analytical models were developed using a simplified fibre element model.

A critical review of RC wall design and detailing in Australia was performed, which included a reconnaissance survey of 35 buildings located in various Australian capital cities. The various observed detailing and design approaches are reviewed and outlined. Upper and lower bound values for typical wall attributes from the review are summarised and were used to ensure the experimental test program best represented industry standard construction practices.

The experimental program included seventeen boundary element test specimens, five large-scale RC wall specimens and three large-scale precast connection specimens. The five large-scale RC wall specimens consisted of two monolithic cast in-situ specimens and three jointed precast building core specimens. The cast in-situ specimens consisted of one rectangular wall and one box-shaped building core specimen. The three precast connection specimens were isolated building core panel-to-panel connections and consisted of one specimen that was an industry standard welded stitch plate connection and then two new prototype connection specimens.

The results of the boundary element test specimens were used to develop a matrix of proposed material strain limits that could be adopted when performing displacement-based assessments of limited ductile RC walls. This testing was also used to develop and validate a tension stiffening model for limited ductile RC elements.

This thesis is concluded with the development of a user-friendly and transparent program for calculating the back-bone force-displacement response of RC walls and building cores. The program was validated against the results of the cast in-situ test specimens and another fourteen test specimens reported in the literature. The program was then used to undertake parametric studies to further investigate the inelastic displacement behaviour of RC rectangular walls and building cores. The results of this parametric study were used to develop simplified empirical models, which are computationally simple and do not require the use of computer-based design aids like spreadsheets, for calculating the lateral load capacity and displacement behaviour of rectangular walls and building cores.
Statement

This is to certify that:

i. This dissertation comprises only my original work.

ii. Due acknowledgement has been made in the text to all other material used.

iii. This dissertation is between 70,000 and 100,000 words in length, exclusive of tables, figures, references, appendices, front matter and footnotes.

Scott J. Menegon
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Declaration

To date, the research findings from this Australian Research Council (ARC) funded project have been presented at ten international conferences and workshops and published in twelve peer-reviewed conference papers, four peer-reviewed journal articles and two book chapters. These papers are listed below with the relevant chapters that have, in part, been used to produce these publications.

Conference papers:


Journal articles:


Book chapters:

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Notation

Except where specifically noted, this thesis uses SI units of kilograms, metres, seconds, pascals and newtons, i.e. kg, m, s, Pa and N respectively. Unless noted otherwise, the notation adopted in this thesis has the following meanings:

- $A_g$ = Gross cross-sectional area of an RC section.
- $A_c$ = Net cross-sectional area of concrete of an RC section.
- $A_{sc}$ = Cross-sectional area of compressive reinforcement in an RC section.
- $A_{sf}$ = Cross-sectional area of reinforcement fitments (i.e. transverse reinforcement).
- $A_{st}$ = Cross-sectional area of tensile reinforcement in an RC section.
- $A_{sv}$ = Cross-sectional area of vertical reinforcement in an RC wall or column.
- $a$ = Cracking spacing.
- $a_{ave}$ = Average cracking spacing.
- $a_{min}$ = Minimum cracking spacing.
- $b_w$ = Compressive flange width of a box-shaped building core.
- $C_h(T)$ = Spectral shape factor with respect to period.
- $c_{UL}$ = Lower characteristic probability value.
- $c_{UL}$ = Upper characteristic probability value.
- $d_c$ = Depth/width of the confined concrete core.
- $d_b$ = Reinforcement bar diameter.
- $d_n$ = Neutral axis depth of an RC section.
- $E_c$ = Modulus of elasticity of concrete.
- $E_s$ = Modulus of elasticity of reinforcement.
- $E_s$ = Inelastic modulus of reinforcement.
- $E_{sec}$ = Secant modulus of elasticity of concrete, i.e. $E_{sec} = f_c'/\varepsilon_{co}$ or $E_{sec} = f_{cc}'/\varepsilon_{cco}$.
- $F$ = Force.
- $F^*$ = Ultimate limit state design force.
- $F_a$ = Response spectrum soil factor – acceleration-controlled region.
- $F_{al}$ = Lateral force corresponding to axial load failure.
- $F_{cr}$ = Lateral force corresponding to the cracking moment being developed.
- $F_{design}$ = Seismic design force for inelastic responding structure.
- $F_{elastic}$ = Seismic design force for equivalent linear-elastic responding structure.
- $F_i$ = Lateral force (e.g. seismic or wind) at the $i$-th level or $i$-th loading point.
- $F_{lf}$ = Lateral force corresponding to lateral load failure.
\( F_u \) = Ultimate force.
\( F_y \) = Yield force.
\( F_{max} \) = Maximum lateral force capacity of structure.
\( F_v \) = Response spectrum soil factor – velocity-controlled region.
\( f'_c \) = Maximum characteristic compressive cylinder stress of concrete.
\( f'_{cc} \) = Maximum characteristic confined compressive stress of concrete.
\( f_{scu} \) = Concrete stress corresponding to ultimate confined concrete strain.
\( f_{cm} \) = Maximum mean compressive cylinder stress of concrete.
\( f_{cmi} \) = Maximum mean compressive stress of in-situ concrete.
\( f_{ct} \) = Maximum characteristic direct tensile stress of concrete.
\( f_{ctm} \) = Maximum mean direct tensile stress of concrete.
\( f_{cu} \) = Concrete stress corresponding to ultimate concrete strain.
\( f_i' \) = Effective lateral confining stress.
\( f_{su} \) = Ultimate stress of reinforcement.
\( f_{sy} \) = Yield stress of reinforcement.
\( f_{sy,L} \) = Lower characteristic yield stress of reinforcement.
\( f_{sy,U} \) = Upper characteristic yield stress of reinforcement.
\( f_{sy,f} \) = Yield stress of reinforcement fitments (i.e. transverse reinforcement).
\( f_t \) = Maximum tensile stress of concrete.
\( G \) = Permanent load (i.e. dead load).
\( H_e \) = Effective height.
\( H_{eff} \) = Effective height of an equivalent 1-DOF system.
\( h_n \) = Height of the uppermost seismic weight or mass or height to the \( n \)-th storey.
\( h_s \) = Inter-storey height.
\( h'_s \) = Clear inter-storey height.
\( h_{s1} \) = First floor inter-storey height.
\( I_{cr} \) = Cracked moment of inertia of an RC section.
\( I_{eff} \) = Effective moment of inertia of an RC section.
\( I_{gross} \) = Gross moment of inertia of an RC section.
\( K_{eff} \) = Effective stiffness of an equivalent 1-DOF system.
\( k^{+ve} \) = Stiffness of an RC element in the positive direction.
\( k^{-ve} \) = Stiffness of an RC element in the negative direction.
\( k_{ave} \) = Average stiffness of an RC element, i.e. \( k_{ave} = 0.5(k^{+ve} + k^{-ve}). \)
\( k_{\text{eff}} \) = Effective stiffness of an RC element.

\( k_p \) = Probability factor.

\( k_s \) = Shear stiff stiffness.

\( k_t \) = Factor for determining building period in accordance with AS 1170.4.

\( k_v \) = Vertical stiffness of a precast panel-to-panel connection.

\( L_b \) = Elastic bond length.

\( L_b' \) = Inelastic bond length.

\( L_p \) = Plastic hinge length.

\( L_{\text{sp}} \) = Yield penetration depth.

\( L_{\text{sy,cb}} \) = Basic development length of a deformed reinforcing bar.

\( L_w \) = Wall length.

\( M \) = Moment.

\( M^* \) = Ultimate limit state design moment.

\( M_i \) = Moment at the \( i \)-th loading point.

\( m_e \) = Effective mass of an equivalent 1-DOF system.

\( N^* \) = Ultimate limit state design axial load.

\( N_{\text{gross}} \) = Theoretical gross compression strength of an RC section, i.e. \( N_{\text{gross}} = f_c' A_g \).

\( N_{\text{max}} \) = Maximum compression strength of an RC section.

\( n \) = Axial load ratio of an element.

\( n_b \) = Axial load ratio corresponding to the balanced point of the section.

\( p_v \) = Ratio of vertical reinforcement, i.e. \( p_v = A_{sv}/A_g \).

\( Q \) = Imposed load (i.e. live load).

\( Q_M \) = The maximum expected design force during a given design life of a structure.

\( Q_N \) = The upper characteristic design force during a given design life of a structure.

\( R_f \) = Force reduction factor used in force-based seismic design, i.e. \( R_f = \Omega \mu \).

\( R_M \) = The actual capacity (i.e. mean strength) of an element or overall structure.

\( R_N \) = The ultimate limit state design capacity of an element or overall structure.

\( S_p \) = Structural performance factor (reciprocal of overstrength, i.e. \( S_p = 1/\Omega \)).

\( s \) = Spacing of transverse reinforcement (i.e. ligatures).

\( T \) = Natural period.

\( T_i \) = Natural period for the \( i \)-th mode of vibration.

\( T_1 \) = Response spectrum first corner period or first mode natural period.

\( T_2 \) = Response spectrum second corner period.
\( t_w \) = Wall thickness.
\( u_b \) = Average reinforcement bond stress across the elastic bond length.
\( u_b' \) = Average reinforcement bond stress across the inelastic bond length.
\( u_{b,f} \) = Reinforcement bond stress after nominal bond stress failure occurs.
\( u_{b,max} \) = Maximum reinforcement bond stress.
\( \ddot{u}_g(t) \) = Ground acceleration with respect to time.
\( V^* \) = Ultimate limit state design shear force.
\( V_D \) = Design shear force for inelastic responding structure.
\( V_E \) = Design shear force for equivalent linear-elastic responding structure.
\( W_i \) = Seismic weight at the \( i \)-th level.
\( x \) = Cracking width.
\( Z \) = Hazard factor.
\( \gamma \) = Lateral drift.
\( \gamma_{af} \) = Lateral drift corresponding to axial load failure.
\( \gamma_{cr} \) = Lateral drift corresponding to the cracking moment being developed.
\( \gamma_{lf} \) = Lateral drift corresponding to lateral load failure.
\( \gamma_p \) = Plastic drift.
\( \gamma_{peak} \) = Lateral drift corresponding to the peak strength being developed.
\( \gamma_u \) = Ultimate drift.
\( \gamma_y \) = Yield drift.
\( \gamma_y' \) = Notional yield drift.
\( \Delta \) = Displacement.
\( \Delta_c \) = Collapse displacement.
\( \Delta_{cr} \) = Displacement corresponding to the cracking moment being developed.
\( \Delta_{eff} \) = Effective displacement of an equivalent 1-DOF system.
\( \Delta_f \) = Flexure displacement.
\( \Delta_i \) = Displacement at the \( i \)-th loading point.
\( \Delta_{n,i} \) = Displacement of the \( n \)-th storey at the \( i \)-th loading point.
\( \Delta_p \) = Plastic displacement.
\( \Delta_s \) = Shear displacement.
\( \Delta_y \) = Yield displacement.
\( \Delta_y' \) = Notional yield displacement.
\( \Delta_u \) = Ultimate displacement.
\( \Delta_v \) = Vertical displacement of precast panel-to-panel connection.
\( e_{\text{ave}} \) = Average strain of a concrete section in tension.
\( e_c \) = Concrete strain.
\( e_{c co} \) = Concrete strain corresponding to maximum confined compressive stress.
\( e_{c o} \) = Concrete strain corresponding to maximum compressive stress.
\( e_{c cu} \) = Ultimate compressive strain of confined concrete.
\( e_{c f} \) = Concrete strain corresponding to failure of the confined concrete core.
\( e_{c u} \) = Ultimate compressive strain of concrete.
\( e_{c s} \) = Concrete strain corresponding to spalling of cover concrete.
\( e_{\text{global}} \) = Global strain of a concrete section in tension (i.e. average strain).
\( e_{\text{local}} \) = Local strain of a concrete section in tension (i.e. reinforcement strain).
\( e_s \) = Reinforcement strain.
\( e_{sp} \) = Yield plateau strain of reinforcement.
\( e_{su} \) = Ultimate strain of reinforcement.
\( e_{su, f} \) = Ultimate strain of reinforcement fitments (i.e. transverse reinforcement).
\( e_{sy} \) = Yield strain of reinforcement.
\( \theta_p \) = Plastic rotation.
\( \mu \) = Ductility factor.
\( \mu_\Delta \) = Displacement ductility.
\( \mu_\phi \) = Curvature ductility.
\( \rho_c \) = Density of concrete.
\( \rho_s \) = Ratio of volume of transverse reinforcement to volume of concrete core.
\( \sigma \) = Standard deviation.
\( \sigma_c \) = Concrete stress.
\( \sigma_s \) = Reinforcement stress.
\( \phi \) = Curvature, or a capacity reduction factor used in ULS design (e.g. \( \phi V_u \) or \( \phi M_u \)).
\( \phi_p \) = Plastic curvature.
\( \phi_{\text{peak}} \) = Curvature corresponding to the peak strength being developed.
\( \phi_u \) = Ultimate curvature.
\( \phi_y \) = Yield curvature.
\( \phi'_y \) = Notional yield curvature.
\( \Omega \) = Overstrength factor.
<table>
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<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tr>
<td>ABCB</td>
<td>Australian Building Code Board</td>
</tr>
<tr>
<td>ADRS</td>
<td>Acceleration-displacement response spectrum</td>
</tr>
<tr>
<td>AEES</td>
<td>Australian Earthquake Engineering Society</td>
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<td>AS</td>
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<tr>
<td>FE</td>
<td>Finite element</td>
</tr>
<tr>
<td>GA</td>
<td>Geoscience Australia</td>
</tr>
<tr>
<td>GPP</td>
<td>Grouted panel pocket (precast building core connection – refer Section 7.3)</td>
</tr>
<tr>
<td>IBC</td>
<td>International Building Code</td>
</tr>
<tr>
<td>IL</td>
<td>Importance level</td>
</tr>
<tr>
<td>LPOT</td>
<td>Linear potentiometer</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear variable displacement transducer</td>
</tr>
<tr>
<td>MCE</td>
<td>Maximum considered earthquake</td>
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<td>National Construction Code</td>
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<td>NZS</td>
<td>New Zealand Standard</td>
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<td>New Zealand Building Code</td>
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<td>Probability density function</td>
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<td>Peak ground acceleration</td>
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<td>Peak ground displacement</td>
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<td>PGV</td>
<td>Peak ground velocity</td>
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<tr>
<td>PSHA</td>
<td>Probability seismic hazard analysis</td>
</tr>
<tr>
<td>PTC</td>
<td>Post tensioned corbel (precast building core connection – refer Section 7.3)</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Definition</td>
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<td>--------------</td>
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</tr>
<tr>
<td>RC</td>
<td>Reinforced concrete</td>
</tr>
<tr>
<td>RP</td>
<td>Return period</td>
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<td>RSa</td>
<td>Response spectrum acceleration</td>
</tr>
<tr>
<td>RSd</td>
<td>Response spectrum displacement</td>
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<tr>
<td>RSv</td>
<td>Response spectrum velocity</td>
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<td>Structural Engineers Association of California</td>
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<td>Shear force diagram</td>
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<td>Serviceability limit state</td>
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<td>Serviceability limit state criteria 2</td>
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<td>String potentiometer</td>
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<td>ULS</td>
<td>Ultimate limit state</td>
</tr>
<tr>
<td>WSP</td>
<td>Welded stitch plate (precast building core connection – refer Section 7.3)</td>
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Chapter 1

Research Background and Objectives

1.1 Research Background

Australia is a region of ‘lower seismicity’ and historically the seismic design and analysis of structures was not required, given little attention or simply ignored. This is in stark contrast to regions of ‘higher seismicity’, e.g. New Zealand, Japan or the west coast of the U.S., where seismic design procedures can be traced back to the 1920s or 1930s [P1]. In Australia, prior to 1979, no earthquake code or document had ever been published nor mandated by the appropriate building authorities for the seismic design of structures in Australia. 1979 saw the publishing of Australia’s first earthquake loading code, AS 2121 [X1]. This document was partially the result of the 1968 magnitude 6.8 earthquake in Meckering, Western Australia.

AS 2121 had only limited implications on the design of structures in Australia, with only the South Australian building code and the then Department of Housing and Construction mandating its use, and Western Australia only requiring its use for buildings in the Meckering area [W1]. AS 2121 was not mandated by the remainder of Australia and hence the majority of buildings constructed in this era would not have been designed for earthquake resistance, like buildings designed prior to 1979. For these reasons it was considered that “AS 2121 has been on the whole, a non-code, ignored largely by the design community and building regulators” [W1].

The attitude of designers and building professionals in Australia shifted following the 1989 magnitude 5.6 earthquake in Newcastle. This event represented a delineation in mindset where interest in earthquake engineering design and research in Australia significantly increased. Coincidentally, the code committee for the earthquake loading standard met about two weeks prior to the Newcastle earthquake [W1], however it was the earthquake itself that created real traction and led to the development of a new earthquake loading standard specifically written for Australia, unlike AS 2121 which was largely a reproduction of earthquake standards from the U.S.

The new Australian earthquake loading code was published in 1993 as Part 4 of the AS 1170 loading code series (subsequently the AS/NZS 1170 loading code series) and denoted AS 1170.4 [X2]. The 1993 version of the code was published in conjunction with a commentary, AS 1107.4
Supp1 [X3]. The general format of the 1993 version of the code laid the way for the now current version of the code published in 2007 [X4] and the commentary which was published shortly thereafter by the Australian Earthquake Engineering Society (AEES) [W2]. The 1993 version of the code was mandated by the Australian Building Code Board (ABCB), but designers could easily avoid designing for earthquake actions if their building was deemed ‘ductile’. Essentially all reinforced and prestressed concrete structures were deemed to be ductile, irrespective of the level of detailing provided, by the 1993 version of the code. The 2007 version requires all buildings to be designed for earthquake actions and was referenced in the Building Code of Australia (BCA) [X5] from around 2008.

This lack of historical perspective on seismic design has resulted in an Australian building stock legacy that has not been designed nor assessed for earthquake resistance. It has also contributed to many misconceptions in the design community regarding seismic design, including the commonly held belief that if the base shear resulting from wind actions exceeds the base shear from an equivalent static seismic analysis, seismic actions can simply be ignored. This misguided belief ignores the fundamental assumption regarding ductility that is made when performing force-based seismic design procedures and has contributed to the adoption of many poor detailing practices in industry, which are discussed further in Chapter 3.

The seismic design mentality in Australia has resulted in many brittle structures that are particularly vulnerable to earthquakes. Poor performance is regularly observed in these types of structures around the world following any significant level of ground shaking. Australian material standards for concrete and steel structures, AS 3600 [X6] and AS 4100 [X7], respectively, are the two most used material standards in the nation. Both standards pay very little attention to seismic design or the notion of achieving a ductile lateral load resisting system.

This has all contributed to the nation having a considerable level of risk in the event of an earthquake. The earthquake risk of a community can simplistically be taken as a function of the earthquake hazard (i.e. a combination of the probability and intensity of an earthquake occurring in the area) and the vulnerability of the building stock (i.e. the strength and ductility of the buildings in the area). While Australia is considered a region of lower seismicity and the earthquake hazard is relatively low compared to higher seismic regions, the earthquake risk is still considerable given the widespread vulnerability of many buildings across the nation. An earthquake in Australia would be considered a low-probability high-consequence event. Walker [W3] claims that earthquake risk “dominates the overall catastrophe insurance risk” in Australia, with an estimated $450 million spent in 2006 by the Australian public on earthquake cover. Walker [W3] continues to surmise that the majority of this money goes “offshore in the form of reinsurance premiums, making Australia one of the largest purchases of earthquake reinsurance in the world”. Separate to this paper, it has also been claimed that premiums in the vicinity of $200 to $300 million are spent annually on reinsurance with overseas companies and “reinsurance companies rate an earthquake in Sydney within their 20 top risk exposures worldwide” [W4].

The 1989 Newcastle earthquake is considered to be the third largest catastrophic event in Australia's history by the Insurance Council of Australia. The 2011 normalised loss of this earthquake is estimated to be AUD$3.2 billion [C1]. The earthquake had a death toll of 13 persons and fortunately occurred over the Christmas New Year’s period, meaning many buildings were unoccupied when the event occurred (e.g. schools). It has been suggested anecdotally that the death toll could have been in the range of 200 to 400 persons if the same event were to have occurred on a ‘business as usual’ day, though the author notes that a reference could not be found to substantiate this claim.
Earthquakes of the equivalent magnitude to the 1989 Newcastle event occur every two to three years and larger events with similar magnitudes to the 2011 Christchurch earthquake occur every ten years somewhere in the Australian continent. The 2011 Christchurch earthquake was a magnitude 6.3 event that caused significant damage to the city, with an estimated 1,000 buildings being partially or fully demolished, a total cost in excess of NZD$40 billion and a death toll of 185 persons, most of which resulted from the complete structural collapse of the CTV tower and the Pyne Gould Corporation Building [H1].

Australia’s many years of poorly regulated seismic design procedures and the lack of emphasis in Australian material standards regarding the importance of embedding ductility into the lateral load resisting system means that the consequences following a similar event to the Christchurch earthquake occurring in a densely-populated region of Australia would be severe. Poor performance, including partial or complete structural collapse, of buildings should not be unexpected.

Reinforced concrete (RC) construction is the dominant form of construction for multi-residential and commercial buildings in Australia and RC walls are used as the primary lateral load resisting system for the majority of low, mid and high-rise buildings. This trend is not limited to Australia and is seen in many other regions of lower seismicity abroad. Traditionally this consisted of either a system of isolated cast-in-situ RC rectangular walls, a central or eccentric cast-in-situ RC building core or a combination of both. The building cores are usually ‘box-shaped’ elements that surround emergency exit stairwells or lift shafts. In recent years, the adoption of precast concrete walls and jointed precast building cores over traditional cast-in-situ RC elements has become increasingly popular in low and mid-rise construction, particularly in the south-eastern Australian states.

Despite their widespread use and functional importance to the structural system of the building, little direct research has been conducted into how RC walls, typical of Australian construction practices, behave and respond under lateral load. In most cases, their design is based on the knowledge and understanding of the fundamental mechanics of RC elements, which has generally been developed and acquired through the experimental testing and research of RC beams, columns and combined moment-frame systems.

Research efforts over the years have generally been more heavily focused towards RC moment-frame structures, which have been observed following many major earthquakes to have a higher inherent risk of collapse. Frame structures without structural walls however, are generally not common in Australia. While the research focus has been towards frame structures generally, there has still been a considerable amount of RC wall research performed internationally over the last 30 to 40 years, but it has generally been performed in regions of higher seismicity and with respect to wall construction not relevant to typical Australian practice. Further, regions of lower seismicity differ from higher seismic regions in two key ways; the first being that the intensity of ground shaking between long and short return period events is much more severe and the second being that the displacement demands are more modest due to the upper magnitude limit of intraplate earthquakes characteristic of low seismic regions. For these reasons, translating research findings or copying design codes and standards from regions of higher seismicity can simultaneously be overly conservative in some aspects, yet inappropriate in others.

The seismic design approach for RC wall buildings in Australia, like many regions of lower seismicity, typically consists of undertaking a force-based equivalent static or pseudo dynamic analysis procedure to ensure local or complete collapse of the structure, and any ensuing loss-of-life, does not occur during a rare (i.e. long return period) earthquake event. Given this is typically
the sole design objective for the building, the condition of the structural system and the building as a whole (i.e. whether it is operational, repairable or non-repairable) following the design earthquake event is usually not considered or of interest. This means most design codes, including AS 1170.4, allow the earthquake actions to be resisted by inelastic response of the buildings lateral load resisting system as a whole.

Force-based design procedures typically consist of analysing the building for a set of externally applied ‘pseudo earthquake forces’, which are equal to the theoretical maximum elastic forces the structure would be subjected to during the design earthquake event divided by a force reduction factor. The force reduction factor is used to account for inelastic response by reducing the seismic design actions the structure is designed for under the premise of the ‘equal displacement approximation’, which assumes the maximum displacement demand of an inelastic responding system is equal to the displacement of an equivalent elastic system with the same initial stiffness. This means the designer, when selecting a force reduction factor greater than 1.0, is explicitly acknowledging that the earthquake actions will be resisted by inelastic response in the structural system (i.e. strength is being traded for ductility). This indirect method for accounting for inelastic behaviour means force-based analysis procedures are largely perceived as elastic design approaches within the design community.

AS 3600 specifies a series of ductility classifications (e.g. ‘limited ductile’, ‘moderately ductile’ or ‘fully ductile’) that have associated design requirements and varying levels of detailing, which if adhered to, should allow a structure to have a given amount of inelastic capacity. Structures are deemed ‘limited ductile’ if the main body of AS 3600 is adopted, ‘moderately ductile’ if the main body and Appendix C of AS 3600 are adopted and ‘fully ductile’ if the New Zealand concrete standard, NZS 3101 [X8] is adopted.

Limited ductile is the primary form of RC construction adopted in Australia. The name limited ductile is somewhat misleading, since in some conditions these elements could be quite ductile, and a more appropriate term would be ‘limited detailing’. Limited ductile RC walls would be equivalent to ‘ordinary walls’ in accordance with ACI 318 [X9]. No direct equivalent to limited ductile exists in NZS 3101, and the equivalent performance would be somewhere between the ‘nominally ductile structures’ and ‘structures of limited ductility’ classifications.

Historically the force behaviour (i.e. strength) of RC elements was the primary focus for research and code development. As such the procedure for calculating the ultimate strength of RC elements is well known (e.g. Park and Paulay [P2], Paulay and Priestley [P1] or Warner et al. [W5]) and codified (e.g. AS 3600 [X6], NZS 3101 [X8], ACI 318 [X9] or EN 1992 [X10]). The ultimate displacement behaviour however, particularly in reference to limited ductile RC walls typical of Australian construction practices, is less understood and known. Recently, research has been performed to better understand and quantify the ultimate displacement behaviour of limited ductile RC columns in Australia and has resulted in the development of a simplified model to predict backbone force-displacement curves [W6, W7, W8, W9].

Higher levels of seismic performance could likely be attained in Australian RC buildings simply through the adoption of better detailing practices and establishing a rationale flexure based (i.e. ductile) yielding mechanism in the major RC elements of the structure. Relatively simple changes to established industry standard detailing practices, where a new emphasis is placed on structural ductility rather than strength, could allow structures to achieve seismic compliance under a much higher hazard without the need for wholesale changes such as larger building cores, additional walls or thicker walls, which invariably would encounter push back by building professionals.
1.2 Research Objectives

The primary objectives of this research project are to develop a better understanding of the inelastic displacement behaviour of limited ductile RC walls and to develop a simplified model for assessing the ultimate displacement (i.e. drift) behaviour of reinforced concrete (RC) walls in regions of lower seismicity, with particular reference to Australia. Furthermore, this thesis aims to improve the seismic design and detailing of RC wall buildings in Australia.

To meet the primary objectives of the project, the following sub-objectives will be addressed:

- Provide a state-of-the-art review of seismic design and assessment procedures currently adopted in Australian design practice.
- Provide a critical review of RC construction in Australia to identify and document typical construction and detailing practices.
- Provide a critical review of RC design and detailing practices in Australia to identify deficiencies and propose improvements.
- Identify local failure mechanisms of limited ductile RC walls and undertake an experimental testing program of prism specimens to inform and develop recommended material strain limits.
- Identify experimental testing programs of RC walls in literature that are relevant to typical Australian construction practices and undertake an experimental testing program of RC walls that will address any gaps in the literature.
- Develop simplified easy-to-use design equations for calculating the non-linear moment-curvature response of limited ductile RC walls and building cores.

This thesis is broadly separated into three parts to address and fulfil the objectives of the project. The three parts are summarised below and then followed by a brief induction:

- Part A begins with an overview and discussion of the seismic design methodology in regions of lower seismicity (Chapter 2), followed by a critical review and discussion of RC wall construction in Australia (Chapter 3) and is concluded with a critical review of the current state-of-the-art knowledge on the displacement behaviour of walls (Chapter 4);
- Part B outlines an extensive experimental testing program conducted on RC walls, which includes boundary element prism tests (Chapter 5), large scale wall tests of monolithic cast in-situ walls (Chapter 6) and large-scale wall tests of jointed precast walls and their connections (Chapter 7); and
- Part C outlines the development of a user-friendly and transparent analysis program for calculating the back-bone force-displacement response of RC rectangular walls and building cores (Chapter 8).

Part A – Design Methodology and RC Wall Design and Construction in Australia

Chapter 2 outlines the current seismic design methodology in regions of lower seismicity, with a particular emphasis on seismic performance objectives and the state-of-the-practice seismic analysis procedures in Australia. The chapter is concluded with a critical review of ultimate limit state seismic design in Australia, where it is argued that the inherent nature of force-based seismic design and the performance objectives specified by the BCA result in a residual seismic risk that is considerable and at odds with other design actions (e.g. wind actions). Chapter 3 provides a critical review and discussion of RC wall construction in Australia. This includes a reconnaissance survey of industry where 35 case study buildings were reviewed in detail. The chapter is concluded with a thorough discussion surrounding many problematic detailing practices that are
currently adopted as pseudo industry standard detailing practices in Australia. A matrix of RC detailing recommendations for various ductility classifications is proposed. Chapter 4 provides a critical review of previous research investigating the displacement behaviour of RC walls and buildings. The emphasis here is placed on the behaviour of limited ductile walls, which are the dominant form of wall construction in Australia. This chapter includes a comprehensive review and summary of experimental RC wall testing programs performed in the last 30 to 40 years internationally.

**Part B – Experimental Testing of Limited Ductile RC Walls and Components**

Chapter 5 provides an overview and results of the first experiment testing program, which consisted of 17 boundary element prism tests, which represented both monolithic cast in-situ construction and precast construction. The specimens had a range of height-to-thickness ratios such that different failure mechanisms could be studied, which included global out-of-plane buckling instabilities and local buckling of vertical reinforcement under reversed cyclic lateral load. The chapter is concluded with the development of a tension stiffening model for limited ductile RC walls, which was validated against the results of the 17 boundary element prism tests.

Chapter 6 provides an overview and results of the second experiment testing program, which consisted of two large scale monolithic cast in-situ test specimens. The first specimen was a rectangular wall and the second was a box-shaped building core specimen. Both specimens were detailed such that they would be classified as limited ductile walls to the Australian earthquake code, AS 1170.4. The specimens were tested under a unidirectional quasi-static cyclic loading regime. Chapter 7 provides an overview and results of the third experimental testing program, which consisted of three ‘system level’ jointed precast building core specimens and three ‘component level’ precast building core connection specimens. The three system level test specimens were jointed precast concrete equivalents of the cast in-situ core tested in Chapter 6. Different reinforcement detailing and connections were used in each specimen. The three component level test specimens consisted of one welded stitch plate connection, which was meant to represent current industry standard construction practices for jointed precast building cores. The second and third component level test specimens were two new types of innovative prototype panel connections, which have been developed to join together precast building cores without site welded connections.

**Part C – Modelling the Displacement Behaviour of Limited Ductile RC Walls**

Chapter 8 outlines the development of a user-friendly and transparent sectional analysis program for predicting the back-bone force-displacement response of RC rectangular walls and building cores. The program is intended to be released as an open-source free-of-charge design tool to assist structural engineers or used as an educational tool for students or researchers. The program can be used for rectangular or non-rectangular cross sections of any geometry. The tension stiffening model developed in Chapter 5 is utilised by the program to account for tension-stiffening in the section when determining the moment-curvature response. The chapter is concluded with a parametric study into limited ductile rectangle walls and building cores. The parametric study was used to develop a series of simple-to-use empirical equations for predicting the non-linear displacement behaviour of limited ductile RC walls.

The conclusions from this research are summarised and presented in Chapter 10, together with recommendations for future research.
Chapter 2

Seismic Design Methodology

Seismic design codes in regions of lower seismicity have historically copied or been heavily influenced by codes of practice in regions of higher seismicity. However low seismic regions differ from high seismic regions in two key ways; the first being the intensity of ground shaking between long and short return period events is much more severe and the second being the displacement demands are more modest due to the upper magnitude limit of intraplate earthquakes, which govern in low seismic regions. This chapter will present these concepts, in addition to the seismic performance objectives and seismic design procedures for a typical region of lower seismicity, i.e. Australia. The chapter is concluded with a conceptual review and philosophical discussion of the ultimate limit state seismic design procedure used for earthquake design in Australia.

2.1 Trending Towards Displacement-Based Seismic Design

Seismic design and analysis of buildings traditionally involved undertaking force-based design procedures, where the building is designed for a set of externally applied forces, which are meant to result in design actions in the structure that would be equivalent to the maximum actions observed during the design level earthquake event. Over the years, limitations and faults with force-based design procedures have been identified and many researchers began exploring new alternative ‘displacement-based’ design and assessment procedures. In the early ’90s Priestley [P3] authored a paper titled Myths and Fallacies in Earthquake Engineering – Conflicts Between Design and Reality that summarised many of the problematic aspects of force-based design.

Priestley, Calvi and Kowalsky [P4] presented a comprehensive argument on the shortcomings of force-based seismic design to support the codification and adoption of a displacement-based seismic design procedure and model code presented therein, where structures would be designed to “achieve, rather than bounded by, a given performance limit state under a given seismic intensity”. The authors of this text note that it is possible to modify current force-based methods to overcome the highlighted shortcomings. However, they surmise that such modifications would result in a more complex procedure that could be ill-suited for codification.
The following summarises some of the flaws and limitations of force-based design presented by Priestley et al. [P4] forming the argument for adopting displacement-based design procedures:

1. **Interdependency between strength and stiffness**: traditionally the member stiffness has been falsely assumed to be independent of strength. Whereas it has been proven that the yield curvature for a given cross section is essentially constant regardless of the reinforcement content and hence, member stiffness is in fact dependent on strength. This is highlighted in Figure 2.1.

2. **Period calculation**: periods estimated with approximate formulae common in seismic codes around the world often underestimate the period and in turn increase the resulting design forces. Additionally, if the period is calculated using the principles of structural dynamics, there can be considerable variation in the period calculated depending on the initial assumptions (e.g. ‘uncracked’ versus ‘lightly cracked’ versus ‘fully cracked’ concrete section properties). Furthermore, as discussed above, element stiffness is dependent on strength, which is not strictly known until the design process is complete.

3. **Ductility factors are assigned based on the structural system**: through the use of various examples presented in [P4], it is shown that the ductility factor can vary significantly for the same structural system depending on the assumptions adopted by the designer. Additionally, there is not a general consensus for the definition of yield and ultimate displacement, which in turn greatly affects the ductility factor – see Figure 2.2(b). This results in different seismic codes around the world having different ductility factors for identical structural systems and materials.

4. **Structural wall buildings with unequal length walls**: it has been proven by extensive research that the yield curvature is inversely proportional to the wall length. Since the yield displacement is directly proportionate to the yield curvature it can be said that walls of different lengths will have different yield displacements. Since the total displacement demand will be the same for all walls in a building during the design earthquake event (ignoring torsional behaviour), yet the yield displacement differs for varying length walls, which means the ductility demand of different length walls will vary.

5. **Force is allocated between elements based upon their elastic stiffness**: this philosophy assumes that different elements can be forced to yield simultaneously, however as highlighted above, in buildings with unequal length walls or additionally in the case of dual wall/frame structural systems, elements will not yield at the same displacement. It would be more appropriate to allocate force based on member strengths (e.g. if walls of different length have a constant reinforcement ratio the force would be distributed based on $l^2_{wall}$ – assuming constant member height and elastic moduli).

6. **Force-based design assumes that increasing the strength of a structure will improve its safety**: increasing the strength of the building will increase the yield curvature (see Figure 2.1(a)) and hence yield displacement, which in turn reduces the ductility demand on the structure. Therefore, the building has a higher level of safety because the ratio of ductility demand to ductility capacity is reduced. Priestley et al. [P4] state that the flaws of this argument are:
   a) The stiffness of an element is not independent of strength, as highlighted previously and shown in Figure 2.1(b);
   b) The equal displacement approximation (see Figure 2.2(a)) is generally not valid and typically non-conservative for short-period structures; and
   c) It is not possible to define a unique ductility capacity (i.e. ductility factor) for a given structural system.
7. **Initial elastic characteristics of the structure are used to give an indication of inelastic performance**; the initial elastic stiffness of reinforced concrete elements is substantially reduced after yielding has occurred. It would be more suitable to use structural characteristics that represented the reinforced concrete elements at maximum response for the limit state being considered.

The general concept of displacement-based seismic design – as an alternative to force-based seismic design and its associated problematic aspects discussed above – has been around for some time in various forms. Displacement-based design could be defined quite broadly as any analysis or design procedure where displacement of the structure, as opposed to force or lateral strength, is used to assess the seismic performance. Research studies were performed generally in the mid-70s by Shibata and Sozen [S1] and mid-80s by Shimazaki and Sozen [S2]. This was followed by research efforts in the late 80s by Priestley and Park [P5] with respect to RC bridge structures and then more generally with respect to both RC frame buildings and RC wall buildings in the early 90s by Moehle [M19], Wallace and Moehle [W10] and Wallace [W11]. In the mid-to-late 90s displacement-based assessment procedures were introduced into seismic evaluation and retrofit standards in the United States, ATC 40 [X11] and FEMA 273/274 [X12, X13].

In 2000, Priestley and Kowalsky [P6] published a journal paper titled *Direct Displacement-Based Seismic Design of Concrete Buildings*. The paper outlined the direct displacement-based design (DDBD) procedure and would later form part of the framework for the text by Priestley et al. [P4] published in 2007: *Displacement-Based Seismic Design of Structures*, which is internationally considered the authority on the subject matter. In 2012 Sullivan, Priestley and Calvi [S3] released a proposed model code on the subject titled *A Model Code for the Displacement-Based Seismic Design of Structures*. The model code is primarily based on the recommendations made by Priestley et al. [P4].

![Figure 2.1](image1.png)  
(a) design assumption: constant stiffness  
(b) realistic condition: constant yield curvature

*Figure 2.1: Influence of strength on yield curvature (reproduced from [P4]).*

![Figure 2.2](image2.png)  
(a) equal displacement approximation  
(b) definition of yield and ultimate displacement

*Figure 2.2: Defining displacement ductility capacity (reproduced from [P4]).*
2.2 Seismic Performance Objectives

The Building Code of Australia (BCA), published as part of the of the National Construction Code (NCC) series [X5], offers two options to achieve seismic compliance, either a ‘deemed-to-satisfy solution’ or ‘performance solution’. The deemed-to-satisfy solution for an RC building requires a designer to determine the relevant earthquake design actions using the Australian Standard for earthquake actions, AS 1170.4 [X4] and then assess the structural resistance of the building using the Australian Standard for concrete structures, AS 3600 [X6]. The performance solution for an RC building allows a designer to work outside the relevant Australian Standards; however due to the vague and non-descript performance criteria set out by the BCA, additional advice from expert consultants would typically be required and generally could not be justified, or seen as impractical, for normal building projects. Prior to discussing the specific performance objectives for Australia, typical seismic performance objectives for regions of higher seismicity will be discussed for context.

2.2.1 Performance Based Seismic Design

A recent development in the seismic design of structures in the last 20 years is to assign seismic performance objectives. Vision 2000 [X14], a document prepared by the Structural Engineers Association of California (SEAOC) in 1995, defined performance objects as the “coupling of expected performance levels with expected levels of seismic ground motions” and further proposed a matrix of seismic performance objects for new buildings. This notion of associating an expected building performance level with an expected level of seismic ground shaking is referred to as ‘performance based seismic design’.

The building performance levels are usually expressed by easy to understand terms such as: level 1, no damage or operational; level 2, repairable damage; and level 3, near collapse or collapse prevention during a maximum considered earthquake (MCE) event. The level of seismic ground shaking is usually expressed by a return period event that is calculated based on a probability of exceedance across the design life of the building. Sullivan et al. [S3] proposed a matrix of seismic performance criteria for buildings and has been summarised in Figure 2.3.

Sullivan et al. [S3] proposes that two zones be established: Zone A, moderate to high seismicity, where Levels 1, 2 and 3 performance criteria shall be met; and Zone B, low seismicity, where only Level 3 performance criteria shall be met. In regions of higher seismicity, it is likely that Level 1 or 2 will govern the design. In regions of lower seismicity where earthquakes are rare and the difference between low and high return period events can be significant (as highlighted later in Figure 2.6), Sullivan et al. [S3] recommend the focus should be placed on solely meeting the Level 3 criteria to ensure loss of life is prevented under long return period events.

There is a general consensus amongst earthquake engineers that an MCE event for normal buildings (i.e. importance level 2 buildings in the Australian context) should have a 2,500-year return period (RP) (rounded up from 2,475-year), which corresponds to a probability of exceedance (PE) of 2% in 50 years (i.e. there is a 2% chance a building with a design life of 50 years would undergo MCE level ground accelerations during its life span). The old version of ASCE/SEI 7-05 [X15], which outlines the minimum design loads for buildings (including seismic provisions) and is referenced by the International Building Code (IBC) for use in California and many other states in the US, specifies that a probabilistic MCE event shall have a PE of 2% in 50 years (i.e. a RP of 2,500 years). It is noted that since the 2012 edition of the IBC [X16] and the release of ASCE/SEI 7-10 [X17], the hazard definition has changed from a PE of 2% in 50 years to a risk-targeted performance requirement where there is a 1% chance of collapse in 50 years.
However, despite this change in hazard definition, the primary intended performance of the former has been retained [X18].

A PE of 2% in 50 years (or 2,500 year RP MCE event has) also been adopted in the National Building Code of Canada [X19] and the UK National Annex to Eurocode 8 [X20]. Furthermore, Tsang [T1] presents a concise argument for adopting a 2,500 year RP MCE event for the seismic design of buildings in Hong Kong and other regions of lower seismicity, which would include Australia. Tsang [T1] argues that a maximum allowable annual fatality risk (i.e. reliability level) of a building occupant during an earthquake should be in the order of the 10⁻⁶ per year and to achieve such a reliability level, a collapse prevention performance objective must be met under a 2,500-year RP MCE event. The commentary to the BCA [X21] says that historically the probability of structural failure has been in the order of 10⁻⁶ per year, which supports the argument by Tsang [T1].

**Building Performance Objective**

<table>
<thead>
<tr>
<th>Design Event</th>
<th>Level 1</th>
<th>Level 2</th>
<th>Level 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQ-I</td>
<td>No damage</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RP = 75 years</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EQ-II</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RP = 225 years</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EQ-III</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RP = 475 years</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EQ-IV</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RP = 1,225 years</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EQ-V</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RP = 2,475 years</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EQ-VI</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RP = 4,975 years</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Unacceptable Performance

**Figure 2.3:** Sullivan et al. [S3] proposed seismic performance objectives.

### 2.2.2 Australian and New Zealand Performance Objectives

The seismic performance objectives for Australia will be discussed alongside the seismic performance objectives for New Zealand. Many Australian standards are published as joint Australian and New Zealand Standards and as such, the two countries share many similarities in terms of design procedures, methodologies, general terminology and compliance procedures. Additionally, the New Zealand city of Christchurch in 2011 experienced a devastating and extremely damaging earthquake that was characteristic of the typical earthquake faulting mechanism and magnitude that could occur in Australia [G1, H1]. The Christchurch earthquake resulted in an estimated 1,000 buildings being partially or fully demolished, a total cost in excess of NZ$40B and a death toll of 185 persons, most of which resulted from the complete structural collapse of the CTV tower and the Pyne Gould Corporation building [H1]. The common terminology and shared design codes, albeit only some, and the hindsight developed regarding the severity of low magnitude ‘bullseye’ events such as Christchurch, make New Zealand an ideal example for contrasting Australia, a region of lower seismicity, against a region of higher seismicity.
The BCA does not provide clear seismic performance objectives for Australian buildings. Instead it provides vague performance metrics that could easily be subject to extremely varying interpretations within the engineering community. The performance requirements from the BCA (Section BP1.1(a)) are summarised below:

a) A building or structure, during construction and use, with appropriate degrees of reliability, must—

i. perform adequately under all reasonably expected design actions; and

ii. withstand extreme or frequently repeated design actions; and

iii. be designed to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage; and

iv. avoid causing damage to other properties,

The vague performance metrics in the BCA – whether intentional or not – impel designers to adopt the prescriptive deemed-to-satisfy approach, in which designers are instructed to use the joint Australian and New Zealand standard for Structural design actions, Part 0: General principles, AS/NZS 1170.0 [X22]. AS/NZS 1170.0 is written as a “means for demonstrating compliance with the requirements of Part B1 of the Building Code of Australia” and establishing “compliance with Clause B1 ‘Structure’ of the New Zealand Building Code (NZBC)”.

AS/NZS 1170.0 generally requires buildings in New Zealand to be designed for two performance objectives: ultimate limit state (ULS) and serviceability limit state (SLS). Importance level 2 and 3 structures require two limit state objectives (i.e. ULS and SLS-1), whilst importance level 4 structures require an additional serviceability limit state (i.e. SLS-2) objective. AS/NZS 1170.0 defines the SLS-1 and SLS-2 limit states as follows:

- SLS-1 – the structure and the non-structural components do not require repair after the SLS-1 earthquake.
- SLS-2 – the structure maintains operational continuity after the SLS-2 earthquake.

The importance level definitions and associated return periods for each limit state objective for buildings in New Zealand are outlined in AS/NZS 1170.0 and have been summarised in Figure 2.4.

**Figure 2.4**: New Zealand seismic performance objectives (AS/NZS 1170.0 [X22]).
AS/NZS 1170.0 requires all buildings in Australia to be designed for ULS earthquake actions based on a given RP for that buildings importance level. The importance level and RP for earthquake actions are both specified in the BCA. The respective seismic hazard for a given RP can then be determined using AS 1170.4. AS 1170.4 also requires designers to perform a ‘special study’ to ensure that importance level 4 buildings remain serviceable (i.e. operational) after the equivalent design event for an importance level 2 building. The seismic performance objectives for Australian buildings are summarised in Figure 2.5.

**Figure 2.5:** Australian seismic performance objectives (BCA [X5]).

It should be noted from Figures 2.4 and 2.5 that for normal buildings (i.e. importance level 2), both the Australian and New Zealand seismic performance objectives, require a ULS design check be performed for only a 500-year RP event. AS/NZS 1170.0 defines ULS as “states associated with collapse, or with similar forms of structural failure”, implying that ultimate limit state in the Australian and New Zealand design context relates to a collapse prevention performance objective. This collapse prevention design approach is ‘somewhat’ in line with what was proposed by Sullivan et al. [S3] for a region of lower seismicity, however the RP event that is being designed for is significantly lower than what is proposed therein and in other international codes, as discussed previously.

Similarly, in New Zealand it appears that for normal buildings the MCE event with a 500-year RP is significantly lower than what it should be. However, it was seen following the serve Christchurch earthquake, which was considered to be in excess of a 2,500-year RP event [G1, K1, T2], that the majority of the building stock achieved a collapse prevention performance criteria. This is due in part to the conservative nature of using force-based (or strength-based) design methods for ensuring collapse of a building does not occur. This is discussed in Volume 3 of the Canterbury Earthquakes Royal Commission [X23] where it is argued that the conservative nature of ultimate limit state design provides the building with adequate reserve strength to achieve adequate performance during a larger event. More specifically, ”the conservative aspects of designing for ULS (that is using the lower characteristics material strengths, strength reduction factors, etc.) gives a structure protection against collapse in an earthquake above the ULS design level of shaking.” These conservative aspects of force-based ULS design and a deeply engrained design philosophy in New Zealand material codes which strives for a fully ductile structural system by adopting strict ‘capacity design’ principles led to adequate performance under what would be considered a ‘real’ MCE event (e.g. an event with a PE of 2% in 50 years or more simply put, a 2,500-year RP event). It should be noted though, that despite neither AS/NZS 1170.0 or the New Zealand earthquake standard, NZS 1170.5 [X24] specifying a 2,500-year RP MCE event, the recent 2017 amendment 3 to the New Zealand concrete standard, NZS 3101 [X8] requires ‘deformation critical’ components to be detailed for a 2,500-year RP event.
Capacity design is design philosophy where the designer “tells the structure how to behave” by establishing relative strength hierarchies in the structural elements. Further, it aims to create such a strength hierarchy in the building that while the maximum lateral load capacity of the structure is approached and exceeded, the building is developing and develops a rational yielding mechanism that allows maximum energy to be absorbed by the failing structure prior to complete structural collapse, e.g. it aims to ensure the development of plastic hinges with desirable flexure failure mechanisms at the base of walls or at the ends of beams in moment resisting frames (i.e. strong column weak beam hierarchy). Capacity design originates from New Zealand and has been implemented and become a fundamentally part of many design codes around the world, albeit typically limited to regions of higher seismicity. Capacity design was originally the brain child of a New Zealand design engineer John Hollings, however it was further developed into its current form by prominent earthquake engineering researchers Robert Park and Thomas Paulay [P7, S4]. It should be noted that while many believe capacity design originated from New Zealand, it is often argued that the concept was in fact was first conceived in the U.S. by Blume, Newmark and Corning [B1]. Regardless, Park and Paulay’s immense contribution and development of the concept cannot be disputed. The reader is directed to [P1, P2] for further detailed discussions on capacity design.

It should be noted that although adequate performance during the Christchurch earthquake was achieved in relation to the criteria set out in AS/NZS 1170.0, the expectations of the design community and the general public were significantly misaligned. The former was well aware that the primary performance objective was to protect life whilst significant building damage being allowed. Whereas the latter expected better performance with lower repair costs following. Meaning the engineering community saw many buildings Buchanan et al. [B2] asserts that following the earthquake there was a broad consensus “that severe socio‐economical losses due to earthquake events, as observed in Christchurch, are unacceptable, at least for well‐developed modern countries like New Zealand”.

It could be argued that Australia has a similar level of ‘conservatism’ built into its ULS design procedures such that a collapse prevention performance objective would also be met if a ‘real’ MCE event were to occur, despite the codes requiring normal building be designed for a mere 500-year RP event. However, there a few distinct differences between Australia and New Zealand, in terms of both design philosophy and seismicity. Australian material standards have no mention of capacity design principles nor incorporate them in any respect. It is the opinion of the author that capacity design was the key ingredient in the New Zealand design philosophy that allowed their buildings to achieve a collapse prevention performance under the much larger than designed level of ground shaking seen during the Christchurch earthquake. Additionally, in regions of lower seismicity like Australia the difference between the current ULS return period event (e.g. 500-year RP event) and an MCE event (e.g. 2,500-year RP event) is much larger than in a region of higher seismicity like New Zealand. This is a widely-known phenomenon and has been illustrated in Figure 2.6.

The Christchurch earthquake was a moderate magnitude intraplate bullseye event, which many Australian seismologists have reported as being the typical type of MCE style event that could occur in any of the major Australian capital cities. It is the opinion of the author that an event of this nature occurring in any Australian city would cause significant damage and result in building stock performance shortcomings far exceeding the expectations of the Australian community. Furthermore, partial or complete structural collapse of buildings, far in excess to what was seen in Christchurch, should not be unexpected in an event of this nature, particularly if located within a soft soil basin.
2.2.3 Recommended Performance Objectives for Australia

The proceeding sections have presented a comprehensive review and discussion into seismic performance objectives for regions of both lower and higher seismicity. Through this it is evident that Australian codes should ideally be rectified such that collapse prevention (i.e. ULS performance) is achieved for a much longer RP event, in the order of 2,500 years for normal buildings (i.e. importance level 2). The author recommends the criteria proposed by Sullivan et al. [S3] (Figure 2.3) be adopted for Australia, where importance level 2 buildings are designed for collapse prevention for an event with a PE of 2% in 50 years and importance level 3 and 4 buildings are designed for an event with a PE of 1% in 50 years. This results in design RPs of approximately 2,500 and 5,000 years respectively for importance level 2 and importance 3 and 4 buildings.

The impact to industry resulting from changes of this magnitude would be significant if current seismic design and assessment approaches were continued to be used. Current design approaches, which consist of force-based seismic design procedures, will inadvertently always under predict the true ULS capacity of a building. Meaning current forced-based (or strength-based) design methods only allow designers to know a conservative estimate of their structures capacity. This is due to the many inherent shortcomings of force-based seismic design, which includes taking conservative estimates of the dynamic properties of the building using overly simplistic empirical formulas (as discussed further in Section 2.1) and assuming a constant level of ductility (i.e. $\mu$ factor) for a given structural system, when the variation can in fact be quite significant depending on the particular design parameters (as discussed in Section 2.1 and illustrated using parametric studies in Section 8.4.4).

By moving away from traditional force-based methods and using displacement-based assessment techniques, in conjunction with the ‘displacement controlled’ phenomenon [L1, L2], a multi-tier displacement-based design procedure could be developed for Australia that practicing engineers could quickly learn, adopt and use to check the seismic compliance of buildings in Australia. A procedure of this nature would more accurately predict the real behaviour and performance of
the building as it does contain the inherent shortcomings associated with force-based design. Meaning the buildings that are truly at risk and vulnerable during a seismic event could easily and quickly be identified with appropriate resources directed accordingly.

The reality is such that the majority of buildings in Australia, due to our generally high standard of construction and low seismic nature of the continent, would in fact have sufficient inherent reserve strength to achieve a collapse prevention performance under very long return period events. Current force-based design procedures are unable to make this assessment. A displacement-based design procedure would allow for an efficient and more accurate method for checking the seismic compliance of buildings under longer more appropriate RP events. Development of such displacement-based methods is at the core of this research.

2.3 Australian Seismicity and the Displacement Controlled Phenomena

Australia is located centrally on the Indo-Australia tectonic plate (Figure 2.7). The Indo-Australian plate is predominantly in a state of compression [L3]. Australia’s central location on its tectonic plate means its seismicity is governed by intraplate earthquakes resulting from fault sources within the tectonic plate itself, which in itself would be the defining criteria for making Australia a region of lower seismicity. This is in contrast to regions of higher seismicity where interplate earthquakes, occurring at the boundary of adjacent tectonic plates, dictate the seismic activity.

The earthquake history in Australia over a 50-year period from the 1st of January 1968 to the 31st of December 2017 is shown in Figure 2.8 (data sourced from Geoscience Australia [X25]). Historically, a magnitude 6 and higher event has occurred about once every 8 to 10 years within the Australian continent (refer Figure 2.8(d)). A magnitude 5 and higher event however, occurs much more regular, with about 2 to 3 events per year (refer Figure 2.8(c)). Fortunately, the majority of larger 5 to 7 magnitude events have historically not occurred near capital cities.

Figure 2.7: World stress map with Australia and the Indo-Australian tectonic plate highlighted (modified from [H2]).
Chapter 2: Seismic Design Methodology

Figure 2.8: Earthquake history in Australia over a 50-year period from 01/01/1968 to 31/12/2017 (data sourced from Geoscience Australia [X25]).

The Australian seismic hazard map has been constructed using a probabilistic seismic hazard analysis (PSHA) approach, similar to what is used in regions of higher seismicity. The use of PSHA in Australia has been the subject of much criticism, particular due to the very limited amount of historical data available. The seismic hazard modelling procedure in Australia is very much a ‘reactive’ modelling process. The hazard map only represents active known faults and as earthquakes occur on previously unknown faults in the continent, the hazard map is updated accordingly.

Tennant Creek, located in the central/northern part of the Australian continent, experienced one of the largest earthquakes on record in Australia. The earthquake was a magnitude 6.6 and occurred in 1988. Prior to this event Tennant Creek was thought to be one of the most stable areas of the continent and was assigned the minimum seismic hazard used at the time. Following this event Tennant Creek now has the highest seismic hazard of mainland Australia (as illustrated in Figure 2.9).
It should be noted that even in regions of higher seismicity where much historic data is available, the use of PSHA is still questioned. Recently, Mulargia, Stark and Geller [M20] performed a comprehensive review of the PSHA approach and makes “the case that PSHA is fundamentally flawed and should there be abandoned” and states that their “concern is not with the numerical inputs used by various PSHA practitioners, but with the methodology itself”.

2.3.1 Seismic Hazard for Building Design

The current seismic hazard for design adopted in AS 1170.4 is based on the 1991 Geoscience Australia (GA) seismic hazard map [M21], which was primarily developed using the hazard analysis by Gaull et al. [G2]. The seismic hazard for design in AS 1170.4 is referred to as the hazard factor ($Z$) and is equal to the effective peak ground acceleration (PGA) associated with a 1 in 500-year RP event at a given location (Figure 2.10). The hazard factor is modified using a probability factor ($p_r$) when different RP events are required: e.g. importance level 2 buildings, RP equals 500 years; importance level 3 buildings, RP equals 1,500 years; and importance level 4 buildings, RP equals 1,500 years. It can be seen in Figure 2.10 that the current hazard factors (taken from the 1991 hazard map) for most Australian capital cities are larger than the values from the most recent 2012 version of the hazard map [L4]. In contrast, the 2,500-year RP values are somewhat lower than the 2012 values. In the current version of AS 1170.4 the probability factor for a 2,500-year RP event is equal to 1.8, which is considerably lower than what would typically be expected in regions of lower seismicity. Figure 2.11 compares the current AS 1170.4 probability factors to the most recent seismic hazard data from GA [L4], highlighting the difference between the two.

The committee for AS 1170.4 is proposing in the next revision that a minimum threshold seismic hazard for the country be implemented. It is being proposed to be the same as the current design values for Melbourne and Sydney, which is 0.08g, 0.10g and 0.12g for importance level 2, 3 and 4 buildings respectively. The next revision will also include an updated and revised seismic hazard map from GA. The minimum threshold values are being proposed as a means to counter the uncertainties around predicting the seismic hazard for Australia, which as discussed, has traditionally had a ‘reactive’ approach to seismic hazard modelling due largely in part to the lack of historical data available. The proposed minimum threshold values have been endorsed using a novel approach where a “uniform distribution of seismic activity within a broad source zone” is assumed [L5].
Figure 2.10: Seismic hazard map and hazard factor values currently adopted in AS 1170.4 (reproduced and modified from [M21]).

Figure 2.11: Comparison of probability factors between the current AS 1170.4 values and the 2012 Geoscience Australia seismic hazard map data [L4].

2.3.2 AS 1170.4 Response Spectrum

The AS 1170.4 response spectrum is constructed around the peak ground velocity (PGV) for a given site and return period earthquake event. The PGV is determined from the associated PGA value (i.e. $k_pZ$) using an empirical formula (Equation 2.1), which was used in the 1993 edition of the standard [W2]. In Equation 2.1 PGV is in the units of m/s and PGA is in the units of g’s.

$$PGV = 0.75k_pZ \quad \ldots 2.1$$

Where: $k_p =$ probability factor for the given return period

$Z =$ hazard factor for the given site
The response spectrum is constructed for rock sites assuming a constant velocity region between first and second corner periods of 0.3s and 1.5s respectively. The constant velocity region, i.e. the maximum response spectrum velocity \( (RSv_{\text{max}}) \) region, is taken to be 1.8 times the associated PGV value (Equation 2.2) [L6, S5]. The rock site response spectrum is modified using site factors to allow for different subsoil profiles (i.e. \( F_d \) and \( F_v \)) as given in Table 2.1. The response spectrum in AS 1170.4 has been formulated assuming linear elastic behaviour and 5% critical damping. It is expressed by Equations 2.3 to 2.5 and illustrated in Figure 2.12(a). The velocity and displacement response spectrums (Figures 2.12(b) and 2.12(c) respectively) are determined by multiplying the acceleration response spectrum by \( \frac{2}{T} \) and \( \left( \frac{2}{T} \right)^2 \) respectively.

\[
RSv_{\text{max}} = 1.8PGV = 1.8(0.75k_Z) \quad \ldots 2.2
\]

\[
RSa(T) = RSa_{\text{max}} = RSv_{\text{max}}\left(\frac{2\pi}{T_1}\right) \quad T \leq T_1 \quad \ldots 2.3
\]

\[
RSa(T) = RSV_{\text{max}}\left(\frac{2\pi}{T}\right) \quad T_1 \leq T \leq T_2 \quad \ldots 2.4
\]

\[
RSa(T) = RSD_{\text{max}}\left(\frac{2\pi}{T}\right)^2 = \left[ RSV_{\text{max}}\left(\frac{T_2}{2\pi}\right)\right] \left(\frac{2\pi}{T}\right)^2 \quad T \geq T_2 \quad \ldots 2.5
\]

Where:
- \( RSa_{\text{max}} \) = maximum response spectrum acceleration
- \( RSv_{\text{max}} \) = maximum response spectrum velocity
- \( RSD_{\text{max}} \) = maximum response spectrum displacement
- \( T_1 = 0.3 \) seconds
- \( T_2 = 1.5 \) seconds

### 2.3.3 Acceleration-Displacement Response Spectrum (ADRS)

An alternative format for presenting the traditional acceleration response spectrum is the acceleration-displacement response spectra (ADRS). ADRS diagrams are advantageous as they can be used in conjunction with the capacity curve of a structure to directly determine the structures performance under the design earthquake event. This method is often referred to as the capacity spectrum method (CSM) and is presented in Section 2.4.3. The ADRS diagram is formulated by equating the maximum kinetic energy (i.e. \( RSv = RSv_{\text{max}} \) and elastic strain energy of a SDOF structure:

\[
\text{elastic strain energy} = \text{kinetic energy}
\]

\[
0.5 \times \text{force} \times \text{displacement} = 0.5 \times \text{mass} \times (\text{velocity})^2
\]

\[
0.5 \times (\text{mass} \times \text{acceleration}) \times \text{displacement} = 0.5 \times \text{mass} \times (\text{velocity})^2
\]

\[
0.5 \times (m_e \times RSa) \times RSD = 0.5 \times m_e \times RSv^2
\]

\[
RSa = \frac{RSv_{\text{max}}^2}{RSD}; \quad RSa < RSa_{\text{max}} \text{ and } RSD < RSD_{\text{max}}
\]

Figure 2.13 shows a typical ADRS and highlights how the first corner period \( (T_1) \) increases for site classes C, D and E. It is worth noting that when using an ADRS in conjunction with the CSM the acceleration axis can be converted to force by multiplying it by the effective mass of the SDOF substitute structure under consideration. Similarly, the displacement axis can be converted to drift by divided it by the effective height of the SDOF substitute structure.
Figure 2.12: AS 1170.4 response spectrum for a rock site in Melbourne (PGA equals 0.08g).

Table 2.1: AS 1170.4 site factors.

<table>
<thead>
<tr>
<th>Site class</th>
<th>$F_a$</th>
<th>$F_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_e$ – strong rock</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>$B_e$ – rock</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>$C_e$ – shallow soil site</td>
<td>1.25</td>
<td>1.40</td>
</tr>
<tr>
<td>$D_e$ – deep or soft soil site</td>
<td>1.25</td>
<td>2.25</td>
</tr>
<tr>
<td>$E_e$ – very soft soil site</td>
<td>1.25</td>
<td>3.50</td>
</tr>
</tbody>
</table>
2.3.4 Displacement Controlled Phenomena: Regular Buildings

The displacement demand of a structure due to earthquake ground excitation generally increases as the flexibility of the structure increases, i.e. as the buildings natural period increases the displacement demand on the structure increases. However, the displacement demand does not continue to increase indefinitely. If the buildings natural period is equal to or greater than the notional limit known as the second corner period \( T_2 \), the displacement demand becomes constant and for design purposes, will be equal to the maximum response spectrum displacement \( RSD_{\text{max}} \). This is a well-known phenomenon commonly referred to as the displacement-controlled phenomena. This can be seen visually in Figure 2.12(c).

In regions of lower seismicity this behaviour is important since the \( RSD_{\text{max}} \) is quite modest and comparable to the ultimate displacement capacity of many structures. Whereas in regions of higher seismicity the \( RSD_{\text{max}} \) is generally too large to be of any practical importance. This is illustrated in Figure 2.14, which compares the ADRS curve of a typical rock site in Australia and New Zealand with a hazard factor of 0.08g and 0.30g respectively. The ADRS curve for the Australian and New Zealand site is constructed using the AS 1170.4 and NZS 1170.5 response spectrums respectively. Figure 2.14 shows that the site in New Zealand has a maximum acceleration demand, which is only approximately 2.4 times greater than the Australian site, whereas the maximum displacement demand increases by a factor closer to 9. The larger relative difference between the maximum displacement demands is due to the different second corner period of the Australian and New Zealand response spectrum, which is equal to 1.5 and 3.0 seconds respectively.
The displacement-controlled behaviour of the Australian response spectrum can be used to great effect when checking a large existing building stock for seismic compliance. A designer can simply calculate the ultimate displacement of the structure and check it against the value corresponding to $1.5RSD_{max}$ for the site (the 1.5 multiplication factor to $RSD_{max}$ is required by AS 1170.04 when using displacement-based assessment techniques). If the ultimate displacement exceeds the $1.5RSD_{max}$ value then the building is satisfactory for seismic compliance. This can be a powerful ‘first tier’ level of analysis in regions of lower seismicity due to the inherent speed and lack of complex calculations. Figure 2.15 shows the very modest $RSD_{max}$ values for different Australian capital cities and respective subsoil classifications. Seismic hazards corresponding to 0.08g and 0.16g have been highlighted in Figure 2.15. The former value is of importance as it is being proposed as the minimum threshold seismic hazard value for importance level 2 buildings, i.e. the majority of construction (refer Section 2.3.1). The latter value is of importance as it is approximately equals the 2,500-year RP seismic hazard, proposed by the 2012 GA hazard modelling data, for most Australian capital cities (refer Figure 2.10).

The significant difference between the maximum displacement demand seen in the Australian response spectrum compared to that of regions of higher seismicity (Figure 2.14), is due to the maximum earthquake magnitude each respective region is capable of generating. Earthquake magnitude controls the second corner period value, which in turn limits the maximum displacement demand. Lam et al. [L6, L7] developed the Component Attenuation Model for modelling the seismic behaviour of Australia and in turn, proposed Equation 2.6 for the second corner period. A more recent study by Lumantarna et al. [L1] further supports the validity of Equation 2.6. The second corner period in AS 1170.4 was selected using Equation 2.6 and a maximum earthquake of magnitude 7.

\[ T_2 = 0.5 + \left( \frac{M - 5}{2} \right) \quad \ldots \quad 2.6 \]

Where: \( M = \) earthquake magnitude
Figure 2.15: Maximum displacement demands based on the AS 1170.4 response spectrum.

2.3.5 Displacement Controlled Phenomena: Irregular Buildings

When a structure is subject to earthquake ground motions and the centre of mass and centre of stiffness of the structure do not coincide, the structure experiences torsional effects. Torsion can be detrimental to a building and can potentially cause catastrophic failures. Torsional effects can also greatly increase storey drifts at the corners of the building, potentially causing failure of poorly detailed cladding systems and gravity frames, which in Australia, are commonly not detailed to sustain large amounts of inter-storey drift or ductility. Torsion is generally difficult to accurately calculate or adequately design for and hence, the general approach is to try to limit the amount of torsion by reducing the building's eccentricity (i.e. the distance between the centre of mass and centre of stiffness). It is noted that the current philosophy for seismic design is to more appropriately consider the centre of strength, as opposed to the centre of stiffness for the same reasons highlighted in Section 2.1.

Lumantarna et al. [L1] recently conducted a study into the effects of torsion on SDOF structures exhibiting both elastic and inelastic behaviour. The findings of the research indicated a similar concept to the displacement controlled phenomena; where continually increasing the buildings eccentricity does not indefinitely increase the torsional actions on the structure. Lumantarna et al. [L1] found that the peak displacement demand, amplified by torsion, on an element consistently never exceed 1.6 times the RSd_{max} value. Implying that if the displacement capacity of a structure is greater than 1.6 × 1.5 × RSd_{max} for the site, the building is compliant for seismic actions regardless of the buildings eccentricity. This study is ongoing and is currently being extended to research this behaviour in multi-storey buildings, i.e. MDOF structures.

2.4 Australian Seismic Design Procedure

The seismic design approach in AS 1170.4 has been translated from the conventional force-based approach used in regions of higher seismicity. This approach involves analysing the structure as a linear-elastic system subject to a set of equivalent earthquake design forces, which are taken as the elastic forces divided by a force reduction factor. The elastic forces are the maximum forces the building would be subjected to if the structure were to maintain a linear-elastic response under the associated level of seismic hazard being considered.
The force reduction factor \((R_f)\) accounts for inelastic behaviour of the building and consists of two components: overstrength \((\Omega)\) and displacement ductility \((\mu)\). AS 1170.4 uses a structural performance factor \((S_p)\) in lieu of the overstrength factor, which is equal to the reciprocal of overstrength, i.e. \(1/\Omega\). Resulting in the force reduction factor used in AS 1170.4 being taken as \(R_f = \mu / S_p\), which is equivalent to \(R_f = \Omega \mu\).

The force-based approach is based on the 'equal displacement approximation' that assumes the inelastic displacement of the structure approximately equals the linear elastic displacement of an equivalent building system, as illustrated in Figure 2.16. The underlying assumption is that the designer is trading strength for ductility, i.e. the structure is resisting the applied kinetic energy from the earthquake ground accelerations by a combination of both elastic strain energy and the energy associated with inelastic plastic deformation of the structure, as opposed to purely elastic strain energy in the case of a linear-elastic responding system. This assumption is handled explicitly in the design process using the force reduction factor and means that the performance of the building during the design earthquake event is not equal to the displacement corresponding to that of the elastic design model of the building, but rather to a point equal to \(\Omega \mu\) times the displacement response of the elastic design model. This is one of the primary differences between wind actions and seismic actions.

The concept of trading strength for ductility in the seismic design process is well understood and acknowledged by designers in regions of higher seismicity; however, in regions of lower seismicity, this concept is somewhat misunderstood by many designers. This misunderstanding has contributed to a commonly held belief that if the base shear resulting from wind actions exceeds the base shear from an equivalent static seismic analysis, seismic actions can simply be ignored. This belief is both inappropriate and false and has resulted in some poor detailing practices of RC wall buildings in Australia, which in many scenarios, can limit the structures ability to develop ductility, resulting in a brittle structure. These detailing practices have been widely adopted as pseudo industry standards, which are commonly used regardless of the ductility assumptions made during the design process. A detailed discussion of RC wall detailing in Australia, with respect to ductility assumptions, is presented in Chapter 3.

**Figure 2.16:** Idealisation of force-based seismic design procedures.
2.4.1 AS 1170.4 Analysis Procedures – Overview

AS 1170.4 requires different levels of analysis depending on the earthquake design category (EDC) of the building. The EDC is a function of the Importance Level of the building, the seismic hazard and the sub-soil class of the site where the building is located and the overall height of the building. EDC I requires an equivalent static analysis where the lateral force at each storey is equal to 10% of the seismic weight for that respective storey; EDC II requires an equivalent static analysis where the lateral force is determined using dynamic properties of the building and an appropriate response spectrum; while with EDC III, dynamic analysis is required. The procedures for EC I, II and III are outlined in Sections 5, 6 and 7 of AS 1170.4 respectively.

Most designers perform either an equivalent static analysis (i.e. Section 6) or multi-modal dynamic analysis (i.e. Section 7) of the structure, both of which are force-based analysis methods. The latest version of AS 1170.4 allows designers to opt for a pseudo displacement-based assessment method by the inclusion in Clause 6.5 saying "it shall be permissible to determine $\mu$ [ductility factor] and $S_p$ [structural performance factor] by using a non-linear static pushover analysis". While the intent of this statement seems somewhat vague, the commentary [W2] further elaborates saying that in this instance, the capacity spectrum method can be adopted in lieu of using an equivalent static analysis method [for EDC II buildings]. The different methods of analysis allowed in AS 1170.4 are summarised in Table 2.2.

2.4.2 Equivalent Static Analysis

The equivalent static analysis is the typical force-based analysis method described above. An equivalent static seismic base shear is determined by multiplying the weight of the building by the building's pseudo maximum acceleration, i.e. $force = mass \times acceleration$. The pseudo acceleration of the building is determined using the AS 1170.4 response spectrum and the fundamental natural period of the building. This force is then reduced by the force reduction factor (i.e. $R_f = \mu \Omega$) to account for inelastic behaviour of the building and the resulting base shear force is then distributed up the height of the building based on its fundamental natural period. The Australian response spectrum was outlined in the previous section and the fundamental natural period of the building is determined using an empirical formula (Equation 2.7).

$$T_1 = 1.25 k_t h_n^{0.75} \quad \ldots 2.7$$

Where: $k_t = 0.075$ for RC moment resisting frame

$= 0.05$ for RC wall structures

$h_n =$ height from the base of the structure to the uppermost seismic weight or mass

2.4.3 Capacity Spectrum Method (Non-Linear Static Pushover Analysis)

The capacity spectrum method (CSM) is a displacement-based approach for determining seismic compliance of the building. The CSM involves overlaying the capacity curve of the structure, as determined by performing a non-linear static pushover, and the ADRS diagram for the building. The intersection point of the two curves is the performance point of the structure (Figure 2.17). While this a very direct and accurate method for determining the seismic compliance of a building, the adoption of this approach in design offices is limited due to the perceived complexities related to determining the inelastic non-linear system behaviour of an RC building.
Table 2.2: AS 1170.4 analysis methods.

<table>
<thead>
<tr>
<th>EDC I – static analysis</th>
<th>Linear analysis*</th>
<th>Non-linear analysis†</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS 1170.4  Section 5</td>
<td><img src="image" alt="Diagram" /></td>
<td>NA</td>
</tr>
<tr>
<td><strong>EDC II – static analysis</strong></td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>AS 1170.4  Section 6</td>
<td>equivalent static analysis</td>
<td>capacity spectrum method</td>
</tr>
<tr>
<td><strong>EDC III – dynamic analysis</strong></td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>AS 1170.4  Section 7</td>
<td>multi-modal dynamic analysis</td>
<td>non-linear time-history analysis</td>
</tr>
</tbody>
</table>

* Linear analysis methods in AS 1170.4 indirectly accounts for inelastic behaviour using a force reduction factor, i.e. $R_f = \mu \Omega$ or $R_f = \mu / S_p$.

† Non-linear analysis methods in AS 1170.4 directly accounts for inelastic behaviour through the adoption of non-linear material models in the analysis.

Priestley et al. [P4] comment that while this approach can accurately determine and track the formation and plastic rotation of plastic hinges, it does not accurately determine the overall displacement demand on the structure. Chopra, Goel and Chintanapakdee [C2] found that a pushover analysis did not accurately predict the roof displacement in a number of cased study buildings that underwent inelastic deformation. Generally, the roof displacement was underestimated and overestimated for buildings with a high and low displacement ductility respectively, with the bias increasing for higher period structures in both scenarios. In both these studies (i.e. [C2, P4]) the emphasis was on moment resisting frame buildings and, as such, these shortcomings of the capacity spectrum method discussed may not extent to RC wall buildings.

Traditionally a pushover analysis was limited to SDOF systems and hence the analysis procedure was only appropriate for low to medium-rise buildings, which primarily deformed in their fundamental mode of vibration.
In more recent years considerable research has been conducted (Chopra and Goel [C3, C4]; Chopra, Goel and Chintanapakdee [C5]; and Goel and Chopra [G3, G4]) to develop a modal pushover analysis (MPA) procedure. Goel and Chopra [G4], citing previous research where the MPA procedure was used to analysis six case study steel moment resisting frame buildings [G3] and 108 generic moment resisting frames [C5], found that the MPA procedure appeared to be able to capture higher-mode contributions and revealed local story mechanisms not detected by traditional first mode pushover analysis.

![Figure 2.17: Capacity spectrum method.](image)

### 2.4.4 Multi-Modal Dynamic Analysis

The multi-modal dynamic analysis is the ‘upper-tier’ force-based analysis method specified in AS 1170.4. This method involves calculating the equivalent static seismic forces, using an appropriate response spectrum and the dynamic properties of the building, for each mode of vibration under consideration for the structure. Designers must consider sufficient modes of vibration such that at least 90% of the mass of the structure is participating for the direction under consideration. AS 1170.4 then states that the peak forces for each mode must be “combined by a recognised method”. The forces from each mode of vibration are reduced by the force reduction factor, similar to the equivalent static analysis method. It has been found that the inelastic behaviour of the structure is typically limited to the first mode of vibration and hence the force reduction factor should only be applied to the forces arising from the first mode response, i.e. the response from the higher modes should be calculated using $R_f = 1.0$ [P4].

### 2.4.5 Non-Linear Time-History Analysis

Non-linear time-history analysis (NLTHA) is the most sophisticated form of seismic analysis allowed in AS 1170.4. NLTHA involves calculating the response of the structure at discrete time intervals associated to the discrete time intervals of a representative earthquake ground motion for the site. The analysis should be repeated for a suite of earthquake ground motions such that the averaged response spectrum for the suite of ground motions covers all possible modes of vibration for the structure that could be excited during its response.
NLTHA would rarely be used on building projects in Australia due to the inherent complexities. Performing NLTHA on an RC wall building requires both a robust non-linear inelastic material and system behaviour model for RC walls and representative earthquake ground motions. The former is not well understood and is the subject of this thesis. Selecting representative earthquake ground motions in regions of lower seismicity like Australia presents many challenges as earthquakes are rare and usable accelerogram data has not been recorded due to the sparse distribution ground motion recording sites across the country. Further discussion and guidance on the implementation and use of inelastic time-history analyses for structures can be found in [C6, P4, W2].

2.5 Ultimate Limit State Design for Seismic Actions

2.5.1 An Overview of Ultimate Limit State Design

Designers in Australia would typically adopt the deemed-to-satisfy solution in the BCA when undertaking the lateral load analysis and design of buildings. This typically consists of performing an ultimate limit state strength assessment of the building in accordance with AS/NZS 1170.0 for the different lateral loading scenarios required, which for buildings typically includes: wind actions, as outlined in AS/NZS 1170.2 [X26]; seismic actions, as outlined in AS 1170.4 [X4]; and robustness, as outlined in Section 6 of AS/NZS 1170.0 [X22].

AS/NZS 1170.0 defines a ‘limit state’ as “states beyond which the structure no longer satisfies the design criteria” and ‘ultimate limit state’ as “states associated with collapse, or with other similar forms of structural failure”. One could surmise that the definition of ultimate limit state design to AS/NZS 1170.0 is to ensure that the building does not collapse or undergo a similar form of structural failure during what the BCA defines as an ultimate limit state event. It is noted that the BCA does not use terminology ‘ultimate limit state event’ but rather ‘design event for safety’.

The BCA defines a ULS event for a normal building (i.e. importance level 2), for both wind and seismic actions, as a 500-year RP event. The design life of a building is typically assumed as 50 years, however this is not explicitly specified or written in any Australian code or standard. It is noted that a 50 year reference period is mentioned in the commentary to the BCA [X21] and a 50 year notional design life is recommended for buildings in AS 5104 [X27], however both of these are informative, not normative documents. This results in importance level 2 buildings having a 10% probability that their 500-year RP design event is exceeded during the notional 50-year design life. The probability of exceedance (PE) can be calculated using the return period (RP) of the design event and design life (DL) of the building using either Equations 2.8 or 2.9. Equation 2.8 is referred to as the ‘intuitive’ method, while Equation 2.9 calculates the PE using a Poisson’s distribution. Both equations give approximately the same result.

\[
PE = 1 - \left(1 - \frac{1}{RP}\right)^{DL} \quad \ldots 2.8
\]

\[
PE = 1 - e^{\left(-\frac{DL}{RP}\right)} \quad \ldots 2.9
\]
When determining the capacity or strength of an element, the BCA requires the use of “five percentile characteristic material properties”. That is, the design capacities of elements must be calculated using material strengths (e.g. yield stress of reinforcement or maximum compressive stress of concrete) where there is a 5% chance that these strengths will not be achieved. The difference between the actual strength of an RC element and the ULS strength calculated using the characteristic material properties is illustrated in Figure 2.18. The reader is directed to Section 4.4.3 for a further discussion on the characteristic and actual material strengths of concrete and reinforcement. To achieve compliance, the ULS design load \( Q_n \) for the action being considered (e.g. the base shear on the building resulting from lateral wind loads) must be greater than the ULS design capacity \( R_n \) of the building (e.g. the ultimate base shear capacity of the building). The ultimate base shear capacity is the associated base shear the building would need to be subjected to such that the ultimate capacity (e.g. ultimate bending moment capacity) is reached in one or more of the critical members in the lateral load resisting system. When this process is followed, the building should achieve a notional probability of failure \( p_f \) in the order of \( 10^{-3} \) to \( 10^{-4} \) for a 50-year reference period [X21].

The probability of failure can be determined by firstly calculating probability density functions (PDF) for both the design actions and design capacity of the building. The PDF for design actions is dependent on the recurrence rate of the type of loading being considered. The PDF for the design capacity is dependent on many factors, which include for example the variability of material properties achieved during the manufacturing process and workmanship during construction. When the two PDFs are plotting side-by-side, the probability of failure is equal to the area of the overlapping forward tail of the design action (i.e. applied load) probability curve and rear tail of the capacity (i.e. strength) probability curve. This has been illustrated in Figure 2.18. The more conservative the design, i.e. the larger the gap between \( Q_n \) and \( R_n \), the smaller the probability of failure becomes as the overlapping region of the curves reduce.

When the deemed-to-satisfy process is followed, it is assumed that the required annual structural reliability index for the action considered (e.g. wind or earthquake) required by the BCA in Table BV1.1 is achieved or bettered (Table 2.4). The annual structural reliability indices are used to convey the probability of failure for a structural component or connection. The annual structural reliability index \( \beta \) and the probability of failure \( p_f \) are related by Equation 2.10. The relationship between these two parameters is shown in Table 2.3.

\[
\beta = -\Phi^{-1}p_f \quad \ldots 2.10
\]

Where: \( \Phi = \) standard normal distribution

**Table 2.3:** Relationship between reliability index and probability of failure [X27].

<table>
<thead>
<tr>
<th>( p_f )</th>
<th>10^{-1}</th>
<th>10^{-2}</th>
<th>10^{-3}</th>
<th>10^{-4}</th>
<th>10^{-5}</th>
<th>10^{-6}</th>
<th>10^{-7}</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \beta )</td>
<td>1.3</td>
<td>2.3</td>
<td>3.1</td>
<td>3.7</td>
<td>4.2</td>
<td>4.7</td>
<td>5.2</td>
</tr>
</tbody>
</table>
Figure 2.18: Left – force-displacement response of a typical RC element showing the mean strength and ultimate limit state strength. Right – action vs. response and probability of failure.

Table 2.4: Annual structural reliability indices for structural components and connections, National Construction Code Volume 1 (BCA) Table BV1.1 [X5].

<table>
<thead>
<tr>
<th>Importance level</th>
<th>Permanent and imposed actions</th>
<th>Wind, earthquake and snow actions</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.8</td>
<td>3.2</td>
</tr>
<tr>
<td>2</td>
<td>3.8</td>
<td>3.4</td>
</tr>
<tr>
<td>3</td>
<td>3.8</td>
<td>3.6</td>
</tr>
<tr>
<td>4</td>
<td>3.8</td>
<td>3.8</td>
</tr>
</tbody>
</table>

Note: the structural reliability indices shown in this table are for primary structural components and connections whose failure could result in collapse of the building, structure of other property. For other structural components and connections, the target structural reliability indices can be reduced by 0.3.

2.5.2 Ultimate Limit State Design for Seismic Actions

The goal of the NCC and as such, the BCA is “to enable the achievement of nationally consistent, minimum necessary standards of relevant safety” [X5], which can be interpreted to mean for seismic actions, it aims to achieve a nationally consistent uniform performance level and underlying risk of collapse of buildings. It should be noted that no building code can ever truly eliminate building collapses and it is the authors opinion that their function is to minimise this risk to a level that is deemed acceptable by society and ensure it be ‘as uniform as possible’ for the range of buildings it covers.

The aforementioned ULS design process very much lends itself to elastic force-based analysis, for which it was primarily developed. When undertaking a lateral load analysis and designing a building for wind actions, the ultimate base shear design action effect (i.e. \( Q_n \)) applied to the building is separate and independent to the ULS design capacity of the lateral load resisting system
(i.e. \( R_n \)). Whereas when undertaking a force-based seismic design and analysis of a building for seismic actions, the ultimate base shear design action effect is directly coupled to the response of the lateral load resisting system and the building generally. This is due to the underlying assumption of seismic design where strength is being traded for ductility and additionally, that the magnitude of force the building is subjected to by the earthquake is dependent on the dynamic properties of the building, which in turn is dependent on the strength and detailing of the lateral load resisting system.

When calculating the seismic loads acting on a building, the elastic loads are reduced by a force reduction factor (i.e. \( R_f = \Omega \mu \)), as discussed in the previous section. The force reduction factor is made up of two components, the first being overstrength (\( \Omega \)) and the second ductility (\( \mu \)). The overstrength component is an acknowledgement that the building will be stronger than what is assumed in the design (i.e. the design strength is determined using capacity reduction factors and characteristic material strengths). The ductility component is an acknowledgement that the building will undergo inelastic deformations to resist the earthquake loads, as opposed to relying purely on elastic strain energy, which is the case for an elastically responding system, to resist the applied earthquake loads. The force reduction factor is the major difference between wind actions and seismic actions. Further, this force reduction factor in seismic design explicitly implies that the building will undergo plastic deformations to resist the design actions resulting from the earthquake.

The expected response of a building under wind loads is in stark contrast to seismic loads. While AS 3600 [X6] allows ‘plastic design’ methods to calculate the ultimate capacities (e.g. ultimate bending moment capacity) of structural elements, the response of these elements during a wind loading event will, in reality, almost always be linear elastic. This is due to the fact that capacities are calculated using characteristic material properties, strain hardening of reinforcement is ignored and hefty capacity reductions factors (i.e. \( \phi \)), usually less than or equal to 0.8, is applied. Once these factors are considered, the ultimate response calculated in accordance with AS 3600, while adopting ‘plastic design methods’, will normally still result in an elastically responding system for wind actions. Obviously if the design wind load is larger than expected, material properties are lower than specified or construction on site is unsupervised and the structure is different to what is specified, the response may no longer be elastic with plastic deformations occurring.

Simply put, the key difference between wind actions and seismic actions, is that under normal expected conditions, i.e. design loads are not larger than expected, materials are manufactured to have the expected mean strengths and construction on site is well supervised and elements are built correctly, the response of the lateral load resisting system will typically be elastic and inelastic for wind and seismic actions respectively for the same RP event with the same PE. This is illustrated in Figure 2.19(d).

To illustrate this further, consider a mid-rise RC building, which has a symmetrical floor plan and a central core that forms the primary lateral load resisting system for the building (Figure 2.19(a)). Under wind actions, the lateral load distribution is typically rectangular and then under seismic actions, it is typically a triangular distribution (Figure 2.19(b)). The scenario will be considered where the wind actions and seismic actions are such that both result in the same effective lateral load being applied on the equivalent single degree-of-freedom (DOF) system for the building (Figure 2.19(c)). Further, in this example it will be considered that the ultimate capacity of the building will be slightly greater, or approximately equal to, the applied effective wind load and seismic load, i.e. \( F_{\text{wind}}^* = F_{\text{seismic}}^* \) and \( F^* \sim \phi F_u \) (Figure 2.19(c)).
Under this scenario described, one not well versed in earthquake engineering and force-based seismic analysis procedures, would say that the performance of this building under wind actions and seismic actions is equal and as such under both environmental loading scenarios, the probability of failure is equal. However due to the explicit assumptions regarding inelastic behaviour of the building that are used when determining the seismic actions, despite the magnitude of forces for the two loading scenarios being the same, the performance point for seismic actions needs to be increased by the overstrength factor in the force axis and by both the overstrength and ductility factor in the displacement axis (Figure 2.19(d)).

\[ \Gamma^* \leq \phi F_u \]

Where: \( \phi F_u = \min \{ \phi V_u : \frac{\phi M_u}{H_{eff}} \} \)

\( \phi V_u \) = ultimate shear capacity

\( \phi M_u \) = ultimate bending moment capacity

\( \Gamma^* \) = equivalent single DOF system

\( \phi F_u \) = real performance point for seismic actions

Figure 2.19: Comparison of wind and seismic actions for a typical RC wall building.
Due to the inherent nature of how each respective action is designed for (i.e. wind and seismic actions), the probability of failure will automatically be different (Figure 2.20(a) vs. Figure 2.20(b)). Despite this, the BCA says the same annual structural reliability index should be achieved for both wind and earthquake actions, i.e. an index of 3.4 for importance level 2 buildings (Table 2.4). This means one of two things:

1. The annual structural reliability index in Table 2.4 has been calibrated using wind actions and as such the probability of failure for earthquake actions is considerably higher than what the code is endeavouring to achieve, i.e. the code is being unconservative with respect earthquake actions.

Figure 2.20: Action versus response probability density functions for wind and seismic actions.
2. The annual structural reliability index in Table 2.4 has been calibrated using earthquake actions and as such the probability of failure for wind actions is considerably lower than what the code is endeavouring to achieve, i.e. the code is being very conservative with respect to wind actions.

Considering the lack of attention that has historically been given to seismic design in Australia, the former would almost certainly be the case. This would suggest that the intent of the BCA is not being achieved using our current design procedures for seismic actions and alternate approaches need to be considered.

2.5.3 A Fundamental Shift in the Way to Approach Seismic Design in Australia

The current and historical approaches to seismic design in Australia has resulted in the building stock likely having an extremely varying level of vulnerability in the event of an earthquake. This is due to the unpredictable and the apparent random nature of earthquake occurrence in lower seismic regions and the many problematic aspects of force-based seismic design. The former has been discussed in Section 2.3 and the latter throughout this Chapter (i.e. Section 2.1 and 2.5). These factors are also coupled with many years of seismic design being somewhat ‘optional’ for designers to undertake, with the BCA only making it mandatory for all buildings from circa 2008 (as discussed in Section 1.1).

An effective approach to simultaneously rationalise seismic design in Australia and provide a framework for achieving a ‘nationally consistent’ uniform level of safety against earthquake events for future buildings, could be to implement a displacement-based design methodology in conjunction with a risk-targeted performance requirement. The displacement-based design approach adopted could be in the form of the ‘direct displacement-based design’ method set out by Priestley et al. [P4] and Sullivan et al. [S3] and a recommended risk-targeted performance objective would be to achieve or better a 1% chance of collapse in 50 years, as adopted in the current versions of the IBC [X28] and ASCE/SEI 7 [X17].

The hurdle with this approach is defining the earthquake hazard for a respective region of Australia. There is still much debate and uncertainty around the seismic hazard in Australia, for both short return period events (i.e. RP = 500 years) and long return period events (i.e. RP = 2,500 years). If the hazard cannot be accurately predicted to any level of certainty, then any performance objective, whether it be a risk-targeted objective as stated above or simply meeting a performance requirement for a given return period event, could never effectively be achieved.

The author proposes an alternative approach, where no ‘direct’ seismic analysis or design is performed for new buildings and hence, the seismic hazard for the respective site, which cannot be predicted in the first place with any level of certainty, is not considered nor needed. For an RC building this approach would have two stages. The first stage would be to design the building for wind loads, as defined in AS/NZS 1170.2 and a robustness load taken as a lateral load at each storey of the building equal to 5% of the weight of the respective storey. This first stage of design would set the overall layout and configuration of the lateral load resisting system for the building. The second stage would entail detailing each respective RC element of the lateral load resisting system using strict capacity design principles such that a ductile system is achieved so under an overload scenario, which would likely be seen during an earthquake, a desirable flexure based yielding mechanism can be developed. The author notes that an approach of this nature is likely only appropriate for RC wall buildings with lateral load resisting systems that are both ‘regular’ and can be designed to be ductile, and different design rules would be required for buildings that don’t meet these criteria, e.g. buildings with vertically discontinuous walls or squat walls.
2.6 Conclusions

Seismic design in Australia is based around achieving a life-safety performance objective under a relatively short return period earthquake event with no consideration given to capacity design principles in any Australian codes. This has left the Australian building stock vulnerable and susceptible to partial or complete structural collapses under very rare long return period earthquake events.

Using current force-based analysis procedures, designing buildings to resist the much larger earthquake actions associated with very rare long return period events would result in significant impact to industry. It would likely result in wholesale changes such as larger building cores, additional walls or thick walls with increased reinforcement, which invariably would all encounter push back by building professionals. The nature however of force-based ULS design is such that it will inadvertently always under predict the true ULS capacity of a building.

Higher levels of seismic performance could, in many scenarios, likely be attained in RC buildings simply through the adoption of better detailing practices and establishing a rationale flexure based (i.e. ductile) yielding mechanism in the mechanism in the major RC elements forming the lateral load resisting system of the structure. Meaning these wholesale changes mentioned above would not be required. However, it is very difficult to accurately assess this using traditional force-based ULS techniques. Using more direct displacement-based seismic design and assessment techniques, buildings could more easily, quickly and accurately be assessed to determine compliance. The development of displacement-based seismic design and assessment techniques for regions of lower seismicity is at the core of this research.
Chapter 3

Critical Review of RC Wall Design in Australia

Seismic design of buildings is a relatively new concept in Australian design practice and has only been properly codified and required in recent times. This has meant that historically wind actions have been the predominant and critical lateral environmental load for the design of buildings. The design for wind actions in Australian codes and standards is essentially an elastic design approach. This has created a deeply ingrained mindsight of Australian designers where the focus is solely placed on the strength or force capacity of the structure and commonly results in the displacement behaviour being rarely considered. This mindsight can be problematic when seismic actions are concerned, as most force-based seismic design procedures explicitly considers and allows for inelastic response of the structure. This has led to many buildings in Australia where the detailing performed is inconsistent with the ductility assumptions made in the design. This Chapter firstly documents the results of a reconnaissance survey performed by the author into RC construction practices in Australia. An overview of observed standard industry practices for RC wall construction in Australia is provided. The Chapter then presents a critical review of widely observed detailing practices and how they commonly contradict the explicit design assumptions of force-based seismic design. The chapter is concluded with advice and recommendations for designers undertaking force-based seismic design procedures for buildings in Australia or other regions of lower seismicity.

3.1 Reconnaissance Survey Overview and Construction Practices in Australia

A reconnaissance survey of low, mid and high-rise buildings in Australia was undertaken to ensure the design and detailing of the test specimens in the experimental program were representative of standard industry construction practices in Australia. It further served to provide upper and lower bound values for the parametric modelling performed as part of this research project, such that most realistic design scenarios are accounted for in the results.
The focus of the reconnaissance survey was multi-storey buildings and consisted of 35 case study buildings constructed in Queensland and Victoria. All the case study buildings were commercial or residential multi-storey buildings. The buildings were designed by 21 different structural engineering firms, many of which are national or multi-national consultancies. Due to confidentiality and intellectual property restraints no specific buildings details (i.e. building names or addresses) or elements of the structural drawings are stated or directly reproduced. The details in this chapter have been produced by the author and are representative of the detailing styles observed in the buildings.

The majority of commercial and residential multi-storey buildings in Australia (i.e. two or more stories tall) utilise RC walls as the primary lateral load resisting elements of the structure and have suspended concrete floor plates. The former is highlighted in Figure 3.1 which shows all but one of the buildings used either cast in-situ or precast RC walls as part of the lateral load resisting system. The remaining building used a combination of cast in-situ RC walls and a RC moment resisting frame. It is believed that these findings are representative and reflect the trend generally seen amongst multi-storey buildings in Australia. Buildings with RC walls were not deliberately targeted during the survey.

There was considerable variation of building types amongst the 35 case study buildings, which consisted of commercial office, residential, university, school, hospital and laboratory buildings (refer Figure 3.1). There was also substantial variation of building heights as seen in Figure 3.1.

![Figure 3.1: Reconnaissance survey overview.](image-url)
The most popular floor framing systems were either one-way slabs supported on band beams or flat plate slabs. Both of these floor systems, as well a typical precast and composite floor system, are illustrated in Figure 3.2. The band beam systems consisted of cast in-situ band beams with traditionally formed one-way slabs between or one-way slabs supported on metal formwork (e.g. Stramit Condeck). The flat plate slabs were cast in-situ floor plates of constant thickness with flat soffits, which greatly simplifies and reduces the cost of the formwork. The precast floor systems consisted of one-way precast floor panels spanning between either cast in-situ or precast primary beams, which were spanning between the supporting columns. A thin topping slab is poured over the precast floor panels after installation to ‘lock’ all the panels in place and form the floor diaphragm. The composite floor systems consisted of primary universal beams (i.e. I-sections) spanning between the supporting columns and secondary universal beams spanning between the primary beams. A one-way composite floor slab then spans between the secondary beams.

Generally, all the residential buildings had flat plate slabs, presumably to reduce inter-storey floor heights and achieve savings on the overall construction cost. Whereas the commercial buildings, as a result of the required additional ceiling space to hide the larger mechanical services typically required, could utilise a more efficient, yet deeper, band beam system. Only one case study building had a composite floor slab, however it should be noted these floor systems are becoming increasing popular in multi-storey construction in Australia.

**Figure 3.2:** Typical floor plate systems used in Australian multi-storey buildings.
3.2 Reinforced Concrete Walls in Multi-Storey Buildings

The lateral load resisting system of all 35 buildings predominantly consisted of RC walls. The majority of the buildings had cast in-situ walls, with the balance precast concrete construction. Precast concrete walls were in buildings less than 10 storeys, located mostly in Victoria and constructed within the last ten years. In recent years precast concrete walls have rapidly started to replace cast in-situ concrete walls in low and mid-rise buildings, especially in Victoria.

In each of the buildings surveyed different wall cross sections were identified. They were typically an isolated box, bundled box, coupled core, geometric, rectangular or 'U', 'T' or 'L' cross sections, as illustrated in Figure 3.3. In addition, Figure 3.3 summarises the occurrence of each wall type in the buildings. The most popular types of wall cross sections are isolated boxes and bundled boxes (accumulative total of 46%). These wall types are generally the result of having RC walls surrounding lift shafts and stairwells to isolate these elements from the main building, which is a typical requirement for fire regulations in the Building Code of Australia.

The isolated boxes and bundled boxes typical had lintel beams (or header beams) over the doorways as shown in the Figure 3.3. The detailing of lintel beams generally consisted of concentrated top and bottom longitudinal reinforcement with closed ligatures for beam depths less than about 600 mm. For deeper lintel beams, typically greater than about 1000 mm deep, the lintel reinforcement was a continuation of the horizontal and vertical reinforcement in the adjacent full height wall section with additional 'U' bars top and bottom. For lintel beam depths between 600 and 1000 mm, both aforementioned detailing styles were observed. The detailing of coupling beams varied significantly from lintel beams in that they had very heavy top and bottom reinforcement, generally placed in multiple layers with N32 spacer bars between, with multiple closed ligatures with close spacings. These detailing practices are illustrated in Figure 3.4. Diagonal reinforced coupling beams, while being the norm in regions of higher seismicity, were not used in any of the buildings and rarely used in Australian practice.

![Figure 3.3: Typical wall cross sections used in Australian multi-storey buildings.](image-url)
3.3 Detailing Practices of Reinforced Concrete Walls

3.3.1 Cast In-Situ Reinforced Concrete Walls

The detailing of cast in-situ RC walls generally consisted of a continuous layer of uniformly spaced vertical and horizontal bars on each face of the wall (i.e. one vertical and horizontal layer per face). Lap splices were included at the base of the wall and typically, additional ‘U’ bars at wall ends, wall intersections and around openings, as illustrated in Figure 3.5. The reinforcement was typically grade D500N to AS/NZS 4671 [X29]. D500N denotes deformed normal ductility reinforcement with a characteristic yield strength of 500 MPa. In some situations, low ductility mesh (i.e. grade D500L to AS/NZS 4671) was used in the uppers floors of the core walls, presumably for ease and speed of construction. In one of the case study buildings, low ductility mesh was used throughout the core walls from the foundations at ground floor through to the roof. This is a concerning practice even though the Australian Standard for Concrete Structures, AS 3600 [X6] does not restrict such application. The use of low ductility mesh in RC walls is discussed in detail later in this Chapter.

The detailing of RC structures in accordance with the main body of AS 3600 should result in what the earthquake standard (i.e. AS 1170.4 [X4]) calls a ‘limited ductile’ structure. This typically consists of provided a continuous layer of vertical and horizontal reinforcement on each face of the wall, no confinement reinforcement and lap splices located at the base of the wall in the plastic hinge regions, as seen in Figure 3.5. This is in contrast to regions of higher seismicity where RC walls are detailed to achieve a ‘fully ductile’ RC structure. This is achieved by the inclusion of closely spaced confinement reinforcement in the boundary elements, locating lap splices away from plastic hinge regions and the use of capacity design principles. The principles of capacity design are discussed in Section 2.2.2. The two contrasting detailing styles are shown side-by-side in Figure 3.6.

Confinement reinforcement can greatly increase the compressive strain of the concrete in the end regions of an RC wall, resulting in a significant increase in the curvature ductility of the cross section. Traditionally the vertical reinforcement would also be concentrated in the confined boundary elements, with minimal reinforcement in the ‘web’ section of the wall (Figure 3.6). Sritharan et al. [S6], by citing various experimental testing programs of RC walls, suggests better performance can be achieved by distributing a higher portion of the vertical reinforcement across the web section of the wall, similar to standard practice in Australia. However, a minimum percentage of vertical reinforcement is still required in the end regions to ensure distributed cracking occurs (distributed cracking in RC walls is discussed later in Section 3.5.4).
In the case study buildings surveyed, confinement reinforcement that would be effective in providing a ductile response was typically not provided. Additionally, Lap splices were generally located at the base of the wall, in the plastic hinge region, in all the buildings surveyed.

The publication Reinforcement Detailing handbook by the Concrete Institute of Australia [X30] provides best practice guidance to designers for detailing reinforced concrete structures in accordance with AS 3600, but is regularly not followed within industry. Generally, the detailing of boundary elements and wall intersections consisted of cogged bars and/or extra ‘U’ or ‘L’ bars. The reconnaissance survey shows most designers detail RC walls using ‘U’ bars at wall intersections, end regions of walls and around openings (Figure 3.7). Typical examples of the various observed detailing practices for corner intersections, ‘T’ intersections, end regions and top cantilever elements of cast-in-situ RC walls are illustrated in Figure 3.8. It is worth noting that while these details for horizontal reinforcement anchorage might generally be appropriate for Australia, where walls are generally detailed for limited ductility, they would not be compliant with the requirements in higher seismic regions for a ductile response (e.g. NZS 3101 [X8]).
3.3.2 Precast Reinforced Concrete Walls

In recent years there has been a rapid rise in popularity of precast concrete construction in Australia, particularly so in the southern states, such as Victoria, where the colder climate is not as favourable for pouring concrete in-situ. Generally precast wall construction consists of individual rectangular panels, with or without openings, that are cast off site in a precast yard or factory and later transported to site for erection. The rectangular panels are erected on site either in isolation or connected together on site to adjacent panels to form different wall configurations as required. In conjunction with this rapid rise in the popularity of precast walls in recent years, has also come the adoption of many detailing practices that are inconsistent with commonly used ductility assumptions. This is discussed in detail in the latter half of this Chapter.

The detailing of precast panels differs somewhat from cast in-situ walls. Panels are typically detailed with one central or one layer per face of L grade mesh with two additional N grade bars around the perimeter of the panel (i.e. trimmer bars). Extra N grade vertical bars are then added as required by the designer. This is illustrated in Figure 3.9. The wall panels are connected vertically using N grade reinforcing bars as dowels that are inserted into cast in-situ grout tubes at the ends of the panel, which are later grouted in place using high strength cementitious grout. The cementitious grout is typically 20 MPa stronger than the characteristic concrete strength of the wall panel. Typical panel-to-panel vertical connections (i.e. grout tube connections) are shown in Figure 3.10. These grout tube connections are also quite similar to what is commonly used in New Zealand [S7].

Adjacent/perpendicular panels are side connected using welded stitch plate connections, as shown in Figure 3.11(a). The 'stitch plate' connections typically consist of structural steel equal angles or flat bar sections site welded to cast in plates on each adjacent panel. The cast in plates have shear studs welded to the rear side to interlock the two. The stitch plates transfer vertical shear force between panels to allow for composite action to be developed. An alternative connection to the welded stitches are wet joints, which consist of a cast in-situ portion of concrete poured between two adjacent precast concrete panels. Wet joints allow for much larger vertical shear forces to be transmitted between panels however they are not preferred by contractors as they are significantly more expensive and slow the floor-to-floor construction cycle. Typical wet joint details are shown in Figure 3.11(b).
Figure 3.8: Observed detailing practices of cast in-situ RC walls.
Figure 3.9: Observed detailing practices of precast walls.

Figure 3.10: Vertical foundation-to-panel and panel-to-panel grout tube connection.

(a) Observed types of welded stitch plate connections

(b) Observed types of wet joint connection

Figure 3.11: Horizontal panel-to-panel connections.
3.4 Design Parameters of Walls

There are many wall attributes to consider when performing parametric computer studies or designing experimental test specimens of RC walls. It is therefore beneficial to have a benchmark of realistic upper and lower bound values for these attributes to ensure the research is as applicable and useful as possible to industry.

The key attributes of interest for RC walls were identified to be: floor-to-floor height, wall aspect ratio, wall slenderness ratio to AS 3600, axial load ratio, vertical and horizontal reinforcement content and the characteristic concrete strength at ground level (i.e. the plastic hinge region). Most of the buildings had multiple walls at the ground level and hence for each of these parameters there were typically a range of values for each building. To capture this range, the minimum and maximum value for each parameter was recorded. This resulted in a ‘lower bound’ and ‘upper bound’ value for each parameter in each building. A summary of these lower bound and upper bound values has been presented in Table 3.1.

For the lower bound set of values, the mean, the coefficient of variation and the minimum has been presented (Table 3.1). For the upper bound set of values, the mean, the coefficient of variation and the maximum has been presented (Table 3.1). For example, the mean range of vertical reinforcement at the ground floor of the case study buildings was between 0.73% and 1.40%.

| Table 3.1: Lower and upper bound values of the case study buildings wall attributes. |
|---------------------------------|-----------------|-----------------|----------------------------|-----------------|-----------------|-----------------|
|                                | **Lower bound values** | **Upper bound values** |
|                                | Minimum | Mean | COV† | Mean | COV† | Maximum |
| Ground floor height            | 3.00 m  | 4.28 m | 0.199 | 4.28 m | 0.199 | 6.40 m |
| Upper floor height             | 2.80 m  | 3.47 m | 0.148 | 3.47 m | 0.148 | 4.50 m |
| Wall aspect ratio              | 0.87    | 5.00   | 0.832  | 9.80  | 0.487  | 19.10 |
| Wall slenderness               | 1.10    | 7.49   | 0.818  | 16.70 | 0.324  | 26.67 |
| Axial load ratio               | 0.001   | 0.038  | 0.872  | 0.058 | 0.767  | 0.196 |
| Vertical reinf. ratio‡         | 0.0015  | 0.0073 | 0.743  | 0.0140 | 0.654  | 0.0329 |
| Horizontal reinf. ratio‡       | 0.0015  | 0.0057 | 1.574  | 0.0060 | 0.675  | 0.0201 |

* For each case study building the minimum and maximum value for each parameter was recorded. The mean lower bound value and mean upper bound value are the mean values of the aforementioned minimum and maximum values respectively.
† COV denotes coefficient of variation.
‡ These are the ground floor values. The reader should note these values decreased up the height of the building.
The characteristic concrete strength at ground level for each of the buildings is shown in Figure 3.12. It was found that the most common concrete grade specified in walls was N50 (i.e. concrete with a characteristic concrete strength equal to 50 MPa). In the cases where concrete grades greater than 50 MPa were specified, it was always for cast in-situ RC walls. All the precast walls were specified to have N40 or N50 concrete grades, with one exception that was N32.

![Figure 3.12: Characteristic concrete strength of the RC walls in the cast study buildings.](image)

The axial load ratio (i.e. \( n \)) was taken to as the design axial load for seismic actions acting on the wall, determined in accordance with AS 1170.4, divided by the product of the gross cross-sectional area and the characteristic concrete strength of the wall (refer Equations 3.1 and 3.2). The reader should note that because the mean cylinder strength of concrete can be considerably higher than the characteristic cylinder strength – i.e. AS 3600 conservatively recommends a mean strength to characteristic strength ratio between 1.1 to 1.25 and Priestley et al. [P4] recommends a ratio between 1.3 to 1.7 – the actual axial load ratio of the walls would likely be somewhat lower than the values presented in Table 3.1.

\[
N^* = G + 0.3Q \tag{3.1}
\]

\[
n = \frac{N^*}{A_g f'_c} \tag{3.2}
\]

Where:
- \( N^* \) = design axial load for seismic actions on the wall
- \( G \) = permanent load on the wall
- \( Q \) = imposed load on the wall
- \( A_g \) = gross cross-sectional area of the wall
- \( f'_c \) = characteristic concrete strength

The wall slenderness ratio was calculated in accordance with AS 3600 Clause 11.4 by taking the effective height of the wall (i.e. \( H_e \)) divided by the thickness of the wall (i.e. \( t_w \)). The effective height of the wall is taken as the clear storey height of the wall (i.e. \( h'_s \)), i.e. the clear distance between the floors above and below the wall providing out-of-plane lateral restraint, multiplied by an effective length factor (i.e. \( k \)). For one-way buckling the effective length factor is 0.75 for walls where restraint against rotation is provided at both ends and 1.0 for walls where no restraint against rotation is provided at one or both ends. Whilst for two-way buckling the effective length factor is given by Equations 3.3 to 3.5.
Two-way buckling where lateral support is provided on three sides by floors and an intersecting wall:

\[ k = \frac{1}{1 + \left( \frac{h'_s}{3L_1} \right)^2} \]

where: \( k \geq 3 \) but not greater than what is obtained by assuming one-way buckling \( \ldots 3.3 \)

Where: \( L_1 = \) the length between the centre of the adjacent wall providing lateral restraint and free end of the wall

Two-way buckling where lateral support is provided on four sides by floors and intersecting walls:

\[ k = \frac{1}{1 + \left( \frac{h'_s}{L_1} \right)^2} \]

where: \( h'_s \leq L_1 \) \( \ldots 3.4 \)

\[ k = \frac{L_1}{2h'_s} \]

where: \( h'_s > L_1 \) \( \ldots 3.5 \)

Where: \( L_1 = \) the length between the centres of the adjacent walls providing lateral restraint

For the purpose of this study it was assumed that an RC slab alone at the end/s of a wall would not provide rotational restraint to the wall, hence an effective length factor of 1.0 was adopted when assessing the wall slenderness ratio under one-way buckling. This is however a conservative assumption as it is likely in many circumstances an RC slab would have sufficient stiffness to provide enough rotational restraint at the end/s of the wall to warrant using the smaller effective length factor of 0.75 and hence the upper bound wall slenderness ratios presented in Table 3.1 could possibly be reduced by a factor 0.75.

### 3.5 Reinforced Concrete Wall Design to AS 1170.4 and AS 3600

Section 11 of AS 3600 sets out the design and detailing requirements for RC walls and is somewhat limited in its scope, with only five pages dedicated to their design and detailing. Given the key role RC walls play in providing lateral stability to the many (if not most) low to high-rise multi-storey buildings in Australia, it is somewhat surprising the limited guidance AS 3600 provides. This section has essentially been written with respect to elastically responding RC walls (subjected to wind design actions) and consequently some requirements appear somewhat inappropriate for a non-linear responding system, which is the case for force-based seismic design where forces are reduced by the product of the ductility and overstrength factor. For example, the standard allows:

- Lateral forces to be distributed based on individual wall stiffness’s determined from gross cross-sectional properties. At a minimum, forces should be distributed using the cracked section properties of the walls or an effective first yield stiffness. Priestley et al. [P4] goes further and argues that lateral forces should be distributed based on relative strengths (as briefly discussed in Section 2.1).
When walls are designed as columns, designers can neglect the requirement for restraining vertical reinforcement. Walls subjected to inelastic in-plane deformations, which are assumed when ductility values greater than 1.0 are taken, would likely undergo concrete compressive strains, in their extreme compressive fibre, much higher than that of columns under ultimate gravity load conditions which must adhere to this requirement of confinement reinforcement.

3.5.1 Ductility and Overstrength Factors for Force-Based Analysis

Ductility (i.e. $\mu$) and overstrength (i.e. $\Omega$) factors for RC design can either be taken from the tables provided in AS 1170.4 or determined by undertaking a non-linear static pushover analysis of the structure. Essentially there are three ductility classes to choose from for RC structures in the tables provided in AS 1170.4: 'limited ductile', where $\mu = 2.0$ and $\Omega = 1.3$; 'moderately ductile', where $\mu = 3.0$ and $\Omega = 1.5$; and 'fully ductile', where $\mu = 4.0$ and $\Omega = 1.5$. The first ductility class, limited ductile, can be adopted when the structure is detailed in accordance with the main body of AS 3600. The second ductility class, moderately ductile, can be adopted when the structure is detailed in accordance with the main body of AS 3600 and Appendix C of AS 3600. The third ductility class, fully ductile, can only be adopted when the structure is designed and detailed in accordance with the New Zealand earthquake loading code, NZS 1170.5 [X24] and New Zealand concrete standard, NZS 3101 [X8], which includes the systematic adoption of capacity design principles and specifying all reinforcement to be E grade to AS/NZS 4671 [X29]. It should be noted that E grade reinforcement is generally not available in the Australian market and typically requires a special order be made by the supplier. The three ductility classes for RC design in Australia are summarised in Table 3.2.

<table>
<thead>
<tr>
<th>Ductility class</th>
<th>$\mu$</th>
<th>$\Omega$</th>
<th>$R_f$</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limited ductile</td>
<td>2.0</td>
<td>1.3</td>
<td>2.6</td>
<td>The structure is detailed in accordance with the main body of AS 3600 [X6].</td>
</tr>
<tr>
<td>Moderately ductile</td>
<td>3.0</td>
<td>1.5</td>
<td>4.5</td>
<td>The structure is detailed in accordance with the main body and Appendix C of AS 3600 [X6].</td>
</tr>
<tr>
<td>Fully ductile</td>
<td>4.0</td>
<td>1.5</td>
<td>6.0</td>
<td>The structure is designed and detailed in accordance NZS 1170.5 [X24] and NZS 3101 [X8].</td>
</tr>
</tbody>
</table>

It is generally recommended to adopt the limited ductile classification and not blindly adopt higher classifications, such as moderately ductile. One should be well versed with the theory and application of earthquake engineering if the higher moderately ductile classification is used. The moderately ductile classification has a force reduction factor of 4.5, which is 73% higher than the limited ductile classification that has a force reduction factor of just 2.6. This significantly higher force reduction factor does not ‘come for free’ and requires the associated higher level of detailing to ensure this extra ductility and inelastic behaviour can actually be achieved in the structure.
The seismic design Appendix C in AS 3600 is long overdue to be updated and in its current form, does not provide adequate direction for the detailing of RC walls to achieve such a high level of ductility. It is recommended designers who wish to adopt a structure classification of moderately ductile, i.e. \( \mu = 3 \), to generally follow the guidance provided by NZS 3101 for a ‘ductile structure’ or perform a non-linear static pushover analysis to ensure their structure has this level of ductility.

For certain detailing practices, a fourth ductility class is being recommended: ‘non-ductile’, where \( \mu = 1.0 \) and \( \Omega = 1.3 \). This ductility class would include RC walls detailed with either or a combination of low ductile reinforcement, central layers of vertical reinforcement or minimal percentages of vertical reinforcement (i.e. less than about 0.7%); which are all detailing practices that extremely hinder a walls ability to develop ductility, yet are extensively being used in low and mid-rise buildings around Australia. This non-ductile class of walls is discussed further below. It is worth noting that AS 3600 is currently undergoing a major revision with respect to seismic design and the current draft, which has been released for public comment, proposes to include this fourth ductility class and much of what is discussed in the subsequent sub-sections.

### 3.5.2 RC Walls Detailed with a Single Central Layer of Reinforcement

The detailing of cast in-situ RC walls in multi-storey buildings generally consists of two grids of vertical and horizontal reinforcement on each face of the wall. In contrast, tilt up panels – used primary as external and internal firewalls in single storey industry buildings – are generally detailed using one central grid of vertical and horizontal reinforcement, typically in the form of a low ductility welded reinforcing mesh. In each of these applications, the walls behave very differently and the respective typical detailing in each scenario has been developed and adopted accordingly.

Precast walls in multi-storey buildings are however very commonly detailed using one central layer of vertical and horizontal reinforcement. RC walls detailed with one central grid of vertical and horizontal reinforcement can have very poor performance under cyclic in-plane loading seen during an earthquake. This has been shown in experimental laboratory testing and recent earthquakes, with one notable example being the Pyne Gould Corporation Building which collapsed during the 2011 Christchurch Earthquake. The Pyne Gould Corporation Building was a four storey RC building that had a large central building core that was detailed with one central grid of vertical and horizontal reinforcement.

Section 11 Design of Walls in AS 3600 sets out the design procedure and detailing requirements for RC walls. Clause 11.7.3 restricts the use of centrally reinforced RC walls in a number of different scenarios; an extract from the clause is reproduced below:

> The vertical and horizontal reinforcement shall be provided in two grids, one near each face of the wall under any of the following conditions:

a) Walls greater than 200 mm thick.

b) Any part of a wall structure where tension exceeds the tensile capacity of the concrete under the design ultimate loads.

c) Walls designed for two-way buckling (based on Clauses 11.4(b) or 11.4(c)).
This clause in AS 3600 is very explicit in saying that any wall that could be subject to tension exceeding the tensile capacity of the wall (e.g. the applied moment on the wall is greater than the cracking moment capacity of the wall) cannot have one central grid of reinforcement. This clause appears seldom to be met in multi-storey precast construction. An underlying assumption of force-based seismic design is that your structure will undergo inelastic plastic deformations to resist the design level earthquake, which is accounted for using a force reduction factor, as discussed previously in Chapter 2.

When undertaking the equivalent static analysis in AS 1170.4 and assuming the default displacement ductility factor of 2.0, designers are acknowledging the structure will undergo inelastic plastic deformations. Inelastic plastic deformations in an RC wall can only occur after the cracking moment capacity of the wall has been exceeded, hence triggering the requirement of Clause 11.7.3 to have two grids, one per face, of vertical and horizontal reinforcement in the wall.

If the project requires the use of centrally reinforced RC walls, the designer must therefore show the applied moment on each respective wall is less than the cracking moment capacity of that wall using seismic actions determined with a displacement ductility factor of 1.0, i.e. elastic response of the building has been assumed.

It should be noted that in some applications of AS 3600 this clause (i.e. 11.7.3) does not need to be adhered to when detailing walls. However, in these instances walls are required to be designed and detailed as columns which are required to that have bars on each face with closed ligatures.

### 3.5.3 RC Walls Detailed with Low Ductility Reinforcement

AS 3600 generally requires designers to specify either N grade (i.e. normal ductility) or L grade (i.e. low ductility) reinforcement manufactured in accordance with AS/NZS 4671 [X29]. N grade reinforcement comes in the form of grade D500N bars (i.e. deformed straight or coiled rebar with a minimum characteristic yield stress of 500 MPa) and L grade reinforcement comes in the form of grade D500L welded mesh (i.e. deformed bars welded together as a mesh with a minimum characteristic yield stress of 500 MPa). The major difference between D500N and D500L is the minimum characteristic ultimate strain of 5% and 1.5% respectively and the strain hardening ratio (i.e. the bars ultimate stress to yield stress ratio) of 1.08 and 1.03 respectively. AS 3600 generally requires a 0.8 capacity reduction factor for N grade reinforcement and 0.6 capacity reduction factor for L grade reinforcement.

Quite commonly detailing of precast concrete walls consists of either a central layer or two layers, one per face, of L grade mesh, with additional N grade vertical bars as required (refer Figure 3.9). A common design approach when calculating the capacity of RC walls with L grade mesh is to reduce the area of steel provided by the mesh by a factor of 0.8 to account for the more severe reduction factor of L grade mesh compared to N grade rebar required by AS 3600. No consideration is given to the reduced ultimate strain of the mesh. This approach originates from the SRIA design guide for RC slabs detailed with L grade mesh [X31]. While this approach may be appropriate for RC slabs as outlined in the design guide, its translation to RC walls is inappropriate, especially in respect to the seismic design of RC walls where inelastic behaviour (i.e. plastic deformation) of the wall is assumed as part of the analysis procedure.

Comprehensive experimental studies have been undertaken to assess the performance of suspended RC beams and slabs reinforced with L grade reinforcement (e.g. [G5, G6, M22, S8]), however to the authors best knowledge no studies have been undertaken to assess the performance of L grade reinforcement in RC walls subject to in-plane lateral load. The results of
RC beam or slab tests cannot be extrapolated to estimate the in-plane lateral displacement behaviour of walls; however their findings can still provide insight as to how they might perform. Gilbert and Sakka [G5] showed that while RC slabs reinforced with L ductility mesh could "easily meet their strength requirements, the slabs lack robustness, with little ability to deform plastically and to redistribute actions". This would suggest that the use of low ductility mesh in RC walls, where performance metrics are both strength and ductility under seismic actions, is grossly inappropriate.

When designing RC walls for seismic actions, using force-based procedures where explicit assumptions regarding ductility are made, it is acknowledged that the lateral load resisting elements will undergo inelastic plastic deformations. This can only happen after the cracking moment capacity of the wall has been exceeded, allowing the vertically reinforcement to solely resist the tension strains. The vertical reinforcement then begins to yield and strain harden as the ductility in the wall is developed. The reduced ductility and ultimate strain of L grade mesh extremely inhibits the walls ability to develop ductility. This is illustrated in Figure 3.13, which shows a comparison of the moment-curvature response of a typical building core using both D500N and D500L material properties in the analysis.

The curvature ductility of the section is significantly reduced when low ductility reinforcement is adopted. The response was calculated using a fibre-element model as outlined later in Section 8.1. Each respective moment-curvature response curve highlights where the 'characteristic ultimate strain' would (theoretically) be reached and where the 'mean ultimate strain' of the reinforcement is reached. The reader should note the mean ultimate strain of D500N and D500L reinforcement has been determined to be approximately 3.3% and 9.5% respectively (refer Section 4.4.3).

Typical stress-strain curves for N grade and L grade reinforcement samples are shown in Figure 3.16, highlighting the reduced ductility of L grade mesh. Additionally, the significantly reduced strain hardening ratio of low ductility reinforcement inhibits the development of a distributed plastic hinge mechanism, as it doesn’t allow for the plastic tensions strains to be effectively distributed vertically across the plastic hinge region of a wall.

Therefore, similar to the previous example, designers, when detailing walls using low ductility reinforcement, should adopt the proposed non-ductile classification with a displacement ductility factor of 1.0 when determining the seismic actions.

![Figure 3.13: Moment-curvature response of a building core – comparison of D500N and D500L](image-url)
3.5.4 RC Walls Detailed with Low Percentages of Vertical Reinforcement

AS 3600 allows RC walls to be detailed with very small percentages of reinforcement, where the lower limit for vertical and horizontal reinforcement is 0.15% and 0.25% respectively. These percentages have been selected with respect to thermal restraint and crack control requirements and have generally been observed to be sufficient for those purposes. However, when the vertical reinforcement in the end region of a wall is below a certain limit, distributed cracking cannot be developed in the wall resulting in one or two major cracks forming with concentrated plasticity in these locations. This behaviour greatly effects a walls ability to develop ductility. Desirable distributed cracking with distributed plasticity allows for larger in-plane displacements and ductility values to be developed as it limits the local tension strain in the vertical reinforcement, which can be a limiting factor for the displacement capacity of an RC walls. This behaviour is illustrated indicatively in Figure 3.14 where two walls have equal magnitudes of lateral displacement, yet the maximum plastic tension strain in the extreme tensile reinforcement for the wall with distributed cracking (i.e. Figure 3.14(a)) is significantly smaller than the wall with an undesirable single crack (i.e. Figure 3.14(b)), as the latter has all the inelastic behaviour concentrated in one location (i.e. the single crack at the base of the wall).

![Figure 3.14: Strain distribution of the extreme tensile reinforcing bar in an RC wall.](image)

The minimum percentage of vertical reinforcement required in a wall to ensure desirable distributed cracking occurs has received much attention in recent years, particularly following the 2011 Christchurch earthquake, where this problem was commonly observed in RC walls [H3]. Lu et al. [L8, L9, L10] performed a comprehensive investigation, consisting of both experimental laboratory testing and calibrated finite element computer modelling, which highlighted the poor displacement behaviour that results when an RC wall cannot develop distributed cracking. This behaviour was also identified and reported by Hoult, Goldsworthy and Lumantarna [H4] in an independent study using calibrated finite element computer models of RC walls.

This minimum percentage of vertical reinforcement required to ensure distributed cracking can quickly be determined intuitively by examining the tensile strength of the concrete in the end region of the wall (i.e. the boundary element of the wall, refer Figure 3.15) versus the amount and strength of the reinforcement in that respective area. If the tensile strength of the concrete in that region (i.e. $f_{ct}$) is weaker than the yield strength of the reinforcement (i.e. $f_{yy}$), distributed cracking will occur. Using this logic Equation 3.6 can be derived as follows:
\[ F_{\text{reinforcement}} \geq F_{\text{concrete}} \]
\[ A_{\text{st}} f_{sy} \geq A_{c} f_{t} \]
\[ \frac{A_{\text{st}}}{A_{c}} f_{sy} \geq f_{t} \]
\[ p_{v} f_{sy} \geq f_{t} \]
\[ p_{v} \geq f_{t} \]
\[ \iff p_{v,\text{min}} = \frac{f_{t}}{f_{sy}} \]

Where: \( f_{t} = \) tensile strength of the concrete

\( f_{sy} = \) yield stress of the reinforcement

\[ p_{v,\text{min}} = \frac{f_{t}}{f_{sy}} \]

... 3.6

\[ \text{Figure 3.15: Boundary elements of various wall cross sections. The boundary element of a rectangular wall is defined in accordance with Eurocode 2.} \]

The ultimate tensile strength of concrete is usually expressed as either the direct tensile strength or the flexural tensile strength. The latter relates to members where the tensile stresses are due to bending/flexure actions, where concrete can sustain a somewhat higher level of tensile stress due to the varying strain gradient across the depth of the section. This is typically the case for sections that are not exceedingly deep. As the section depth (i.e. wall length, \( L_{w} \)) increases the flexural tensile strength approaches and eventually equals the direct tensile strength. The Eurocode 2 [X10] model proposes this occurs when the section depth is 1600 mm or greater. It would be unusual for walls to have a length less than 1600 mm and as such, for a generalised approach, it would be better suited to use the direct tensile strength.

AS 3600 specifies the ultimate direct tensile strength of concrete to be 0.36 multiplied by the square root of the characteristic compressive strength, i.e. \( 0.36 \sqrt{f_{c}'} \). The code then suggests a factor of 1.8 for the ratio of lower to upper characteristic tensile strength. When calculating the minimum percentage of reinforcement using Equation 3.6 to ensure distributed cracking occurs, adopting the upper characteristic tensile strength of concrete is desirable, given the high variability in concrete mix designs seen on site. This would result in a tensile strength of concrete as follows: \( f_{t} = 1.8 \times 0.36 \sqrt{f_{c}'} \). For typical 500 MPa reinforcement, the minimum percentages of vertical reinforcement required to achieve desirable distributed cracking for various standard grades of concrete are summarised in Table 3.3.
Similar to the first two scenarios, the proposed non-ductile classification with a displacement ductility factor of 1.0 should be adopted when determining the seismic actions if the walls in the building have vertical reinforcement percentages, in the plastic hinge regions, lower than the values presented in Table 3.3.

Table 3.3: Minimum boundary element reinforcement ratios to ensure distributed cracking.

<table>
<thead>
<tr>
<th>Concrete grade: N32</th>
<th>Concrete grade: N40</th>
<th>Concrete grade: N50</th>
<th>Concrete grade: S65</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{v,\text{min}}$</td>
<td>0.7 %</td>
<td>0.8 %</td>
<td>0.9 %</td>
</tr>
</tbody>
</table>

3.5.5 Dowel Connections in Precast Walls

The typical detail for connecting precast concrete wall panels vertically in multi-storey buildings is a grouted dowel connection, as discussed previously (refer Figure 3.10). This detail is most commonly constructed using N grade reinforcing bars as dowels. While not a common practice, it is occasionally observed that the connection is detailed using all-thread for the dowels. All-thread generally comes in two grades, 4.6 (i.e. mild steel) and 8.8 (i.e. high tensile). Grade 4.6 and 8.8 all-thread has a minimum characteristic ultimate tensile stress of 400 and 830 MPa respectively and a yield stress equal to approximately 60% and 80% respectively, of their respective ultimate tensile stress. The mild steel grade 4.6 all-thread has a much higher ultimate strain compared to that of high tensile grade 8.8 all-thread, as shown in Figure 3.16.

Despite the mild steel grade 4.6 all-thread having a very high ultimate strain and strain hardening ratio (i.e. ultimate stress to yield stress ratio) – the two generic factors required for reinforcement in ductile RC elements – it is still ill-suited for using as reinforcement or dowels in RC elements. This is due to the majority of strain hardening occurring over a very small region from about 1.5% to 2.5%, with the remaining much smaller proportion occurring from about 2.5% to 17% (refer to Figure 3.16). This is in stark contrast to D500N grade reinforcement, which has the strain hardening distributed over a much larger range from about 2% to 10% (refer to Figure 3.16). Similarly, high tensile grade 8.8 all-thread is also ill suited for using as reinforcement or dowels in RC elements since it has a much smaller ultimate strain compared to that of D500N grade reinforcement. Recommended best practice for detailing dowel connections in RC walls is to specify D500N grade dowels only.

As discussed previously, when ductility is assumed, it is acknowledged that the wall will undergo tension forces in the end regions (irrespective of what is shown using the elastic analysis model, which is uses the reduced seismic loads as discussed throughout Chapter 2, refer Figure 2.16 and 2.19). Therefore, when ductility is assumed in the design, the total area of dowel bar steel at the base of the wall has to be equal to or greater than the area of vertical steel provided in the wall. If not, an adequate load path that is consistent with the assumptions made in the analysis process regarding ductility has not been provided. In summary, if the dowels in precast construction are specified to be grade 4.6 or 8.8 all-thread and or specified as D500N bars but having a total area of steel less than the vertical reinforcement in the wall, then the proposed non-ductile classification with a ductility factor of 1.0 should be adopted.
Figure 3.16: Typical stress-strain curves of various reinforcement and all-thread grades.

3.5.6 Overall System Behaviour and Precast Walls as Gravity Load Only Elements

Multi-storey buildings in Australia, particularly apartment buildings, are commonly constructed using a central or eccentric cast in-situ building core and rectangular RC precast walls around the perimeter of the building. These perimeter walls typically support the floor slab and are often referred to as ‘gravity load carrying only elements’ and detailed with a central layer of L grade reinforcing mesh. The cast in-situ core is assumed to take all the lateral load. However, these perimeter walls are tied into the floor diaphragm and as such will take lateral load. Therefore, consideration of the different points discussed above in Sections 3.5.2 to 3.5.5 is still required. It would be a false assumption to assume these walls are gravity load carrying only elements, as an earthquake will distribute lateral load to all elements linked by the floor or roof diaphragm based on their respective in-plane strength and stiffness, regardless of their common held description. Behaviour of the overall system must be considered. Further, this is explicitly required in AS 1170.4 where it is noted in Clause 5.2.3 that these elements must be “considered to be part of the seismic-force-resisting system and designed accordingly”.

Often the lateral load is distributed between elements (e.g. walls and or cores) based on the initial gross second moment of inertia value of the RC element. If the seismic loads are determined using a ductility factor of 2.0 (or higher) and it is shown that these perimeter walls take minimal lateral load, it would still be false to ignore the considerations discussed in Sections 3.5.2 to 3.5.5. This approach would not account for the reduced stiffness of the core after (a) the cracking moment capacity of the core is exceeded and (b) the inelastic plastic deformation required to develop the ductility factor of 2.0. The overall behaviour of the building must be considered to ensure displacement capability of the overall structure is achieved. The lateral load should be distributed based on the effective (i.e. first yield or ‘cracked’) section properties of each element and it should be emphasised again (as discussed in Section 2.4), that the actual displacement or performance point of the structure is equal to a point equal to $\Omega \mu$ times the displacement response of the elastic design model (as shown in Figure 2.19(d)). All elements in the structure, including non-structural elements, must be designed and detailed such that they perform adequately when subjected to this amount of lateral displacement.
Torsional effects must also be considering when the overall system behaviour of a building or structure is being assessed. Torsional raking effects will be induced in the building if the centre-of-stiffness and centre-of-mass of the building do not align during seismic actions. Due to the inherent uncertainties in calculating the centre-of-mass AS 1170.4 requires the earthquake actions to be applied at a position that is either $\pm 0.1b$ from the calculated centre-of-mass, whichever gives the worse effect, where $b$ is equal to plan dimension of the structure perpendicular to the direction of loading being considered. Designers should attempt to minimise the distance between the centre-of-stiffness and centre-of-mass of the building to reduce the possible amount of torsional behaviour under seismic actions.

### 3.5.7 Axial Load Ratio

The simplest and most effective method for increasing the displacement capacity of RC elements is to reduce the axial load ratio (i.e., Equation 3.2) of the column or wall. This was highlighted in a recent study by Wilson et al. [W9], where a simplified model for predicting the maximum drift capacity of RC columns was developed. An example has been provided in Figure 3.17 to illustrate this behaviour. In this example, the lateral drift at axial load failure (i.e., complete structural collapse) is reduced by a factor of 2.3 when the axial load ratio is increased from 10% to 30%.

Alarcon, Hube and Liera [A1] performed an experimental study where three identical walls with different axial load ratios were tested under quasi-static cyclic lateral load until failure. Axial load ratios of 15%, 25% and 35% were used for the three specimens. The specimens failed at 2.7%, 1.8% and 1.5% lateral drifts respectively. In this study, it was shown that the drift capacity of the wall decreased by a factor of 1.8 when the axial load ratio is increased from 15% to 35%.

Shegay et al. [S9] also performed an experimental study where four rectangular walls with different confinement detailing and axial load ratios were tested under quasi-static cyclic lateral load until failure. In this study, the measured plastic rotation at the base of the wall was reduced by a factor of about 1.6 (from 3.3% to 2.1%) when the axial load ratio was increased from 10% to 20%. Both of these studies show good agreement with the model presented by Wilson et al. [W9] and Figure 3.17.

Based on these studies, it is recommended to limit the axial load ratios on RC walls to below 20% in all scenarios.

![Figure 3.17: The effect of axial load on RC columns.](image)
3.6 Recommendations for Force-Based Seismic Design

The recommendations made throughout the second half of this Chapter have been summarised for the convenience of the reader in Table 3.4. Additional notes are also provided to assist in implementing these recommendations into design scenarios. Where reference to NZS 3101 is made, the intent is that the recently released amendment 3 version of the code is adopted. Additional items that were not discussed in this Chapter that designers should consider when performing seismic analyses of RC wall buildings including:

- Slenderness ratios of walls, i.e. height-to-thickness ratios, to prevent out-of-plane instabilities under reversed lateral load. This behaviour is discussed further in Section 4.3.4.2. Currently no firm guidance is provided in literature, however there are many ongoing research effects currently being undertaking, e.g. [D1, R1]. In the meantime, Wallace [W12] has recommended limiting the $h_w'/t_w$ to ratio to 16 or less, where $h_w'$ is the inter-storey unrestrained height of the wall. The 2014 edition of ACI 318 [X9] has introduced this slenderness ratio limit of 16. Wallace [W12] also precautions that a value of 16 may still be insufficient to prevent out-of-plane instabilities in some wall configurations, however until more comprehensive studies are completed, a $h_w'/t_w$ limit of 16 is being recommended for plastic hinge regions or elsewhere in RC walls where significant inelastic plastic tensions strains can be developed.

- Ensuring a desirable flexure failure occurs and not an undesirable brittle shear failure, i.e. shear behaviour of the wall should not be allowed to control the strength of the wall. This could be achieved by designing for a minimum shear force value that corresponds to the actual flexural moment capacity of the wall, i.e. multiply the design shear force by a value corresponding to the actual moment capacity of the wall divided by the design moment acting on the wall. Priestley et al. [P4] proposes that the overstrength for an RC element, where the capacity has been calculated using characteristic material properties and no allowance for strain-hardening in the reinforcement, can be taken as 1.6. Therefore, $\phi V_{ut} \geq (1.6M_{ut}/M_*) \times V^*$. This approach does not account for shear amplification in taller high-rise buildings, however the NZS 3101 approach could adopted in this instance.

- Finally, some building configurations can result in walls having very low aspect ratios or similarly, low shear-span ratios. It should be noted that the latter can be a result of coupling between adjacent elements rather than the physical properties of the individual wall. In these scenarios, it is recommended to design the wall as a non-ductile element using strut and tie methods, as proposed in Table 3.4.
### Table 3.4: Summary of recommendations for RC wall detailing in Australia.

<table>
<thead>
<tr>
<th></th>
<th>Non-Ductile</th>
<th>Limited Ductile</th>
<th>Moderately Ductile</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Ductility factor, ( \mu )</strong></td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td><strong>Overstrength factor, ( \Omega )</strong></td>
<td>1.3</td>
<td>1.3</td>
<td>1.5</td>
</tr>
<tr>
<td><strong>Structural performance factor, ( S_p )</strong></td>
<td>0.77</td>
<td>0.77</td>
<td>0.67</td>
</tr>
<tr>
<td><strong>Force reduction factor, ( R_f )</strong></td>
<td>1.3</td>
<td>2.6</td>
<td>4.5</td>
</tr>
<tr>
<td><strong>Low ductility reinforcement</strong></td>
<td>Permitted</td>
<td>Not permitted</td>
<td>Not permitted</td>
</tr>
<tr>
<td><strong>Centrally reinforced walls</strong></td>
<td>Permitted</td>
<td>Not permitted</td>
<td>Not permitted</td>
</tr>
<tr>
<td><strong>Vertical reinforcement ratio in plastic hinge regions of the wall(^*)</strong></td>
<td>Current code provisions</td>
<td>As given in Table 3.3(^\dagger)</td>
<td>As given in Table 3.3(^\dagger)</td>
</tr>
<tr>
<td><strong>Confinement (cross tie) reinforcement</strong></td>
<td>Current code provisions</td>
<td>Current code provisions</td>
<td>NZS 3101 provisions(^\ddagger)</td>
</tr>
<tr>
<td><strong>Splicing of vertical reinforcement in plastic hinge regions of the wall(^*)</strong></td>
<td>Lap splices permitted</td>
<td>Lap splices permitted</td>
<td>NZS 3101 provisions(^\ddagger) or mechanical splices(^\S)</td>
</tr>
<tr>
<td><strong>Development of horizontal reinforcement(^||)</strong></td>
<td>No requirement</td>
<td>Required at ends of walls and all wall intersections</td>
<td>Required at ends of walls and all wall intersections</td>
</tr>
<tr>
<td><strong>Axial load ratio(^#)</strong></td>
<td>( \leq 0.20 )</td>
<td>( \leq 0.20 )</td>
<td>( \leq 0.20 )</td>
</tr>
<tr>
<td><strong>Shear design</strong></td>
<td>No requirement</td>
<td>( \phi V_u \geq \left( \frac{1.6 M_u}{M_m} \right) V^* )</td>
<td>( \phi V_u \geq \left( \frac{1.6 M_u}{M_m} \right) V^* )</td>
</tr>
<tr>
<td><strong>Slenderness ratio in plastic hinge regions of the wall(^*)</strong></td>
<td>Current code provisions</td>
<td>( \frac{h_s}{t_w} \leq 16 )</td>
<td>( \frac{h_s}{t_w} \leq 16 )</td>
</tr>
<tr>
<td><strong>Aspect ratio(^\¶)</strong></td>
<td>No requirement</td>
<td>( \frac{H_w}{L_w} \geq 2 )</td>
<td>( \frac{H_w}{L_w} \geq 2 )</td>
</tr>
<tr>
<td><strong>Precast construction</strong></td>
<td>Permitted</td>
<td>Permitted</td>
<td>Permitted</td>
</tr>
<tr>
<td><strong>Dowel bars in precast grout tube connections</strong></td>
<td>No requirement</td>
<td>D500N bars only</td>
<td>D500N bars only</td>
</tr>
<tr>
<td><strong>Dowel area in precast grout tube connections</strong></td>
<td>No requirement</td>
<td>( A_{st,dowel} \geq A_{st,wall} )</td>
<td>( A_{st,dowel} \geq A_{st,wall} )</td>
</tr>
</tbody>
</table>

\(^*\) The plastic hinge region of walls in first mode dominant structures is to be taken as the bottom two storeys of the building or twice the depth of the member, whichever is greater. Designers should consider higher mode effects and the development of secondary hinges up the height of the wall in slender walls.

\(^\dagger\) The vertical reinforcement ratio can be decreased at a rate of 15% per storey away from the plastic hinge regions.

\(^\ddagger\) The NZS 3101 provision for ductile RC walls should be adopted. The moderately ductile class, in the Australian context, corresponds to a level of performance between NZS 3101 classes 'structures of limited ductility' and 'ductile structures'. Therefore, the provisions of the higher, ductile structures class should be adopted.

\(^\S\) Mechanical splices to be bar break systems only, i.e. under tensile load the bar shall fracture either side of the mechanical splice and be able to sustain inelastic strains across multiple cycles. The mechanical splices shall also be staggered vertically by a length equal to the standard lap length of that bar.

\(^\|\|\) e.g. additional 'U' bars lapped with horizontal reinforcement at wall intersections and end regions of walls or caging horizontal reinforcement into confined end regions of walls (refer Figure 3.6).

\(^\#\) Axial load ratio is equal to \( N^*/(f_c' A_g) \) where \( N^* \) is calculated using the load combination for earthquake actions given in AS/NZS 1170.0 and AS 1170.4.

\(^\¶\) Walls with an aspect ratio less than 2 should be designed as non-ductile using strut and tie methods.
3.7 Conclusions

A reconnaissance survey of the Australian RC construction industry has been performed. The survey involved the review of the structural documentation of 35 case study buildings, designed by 21 different structural consultancy firms and constructed in either Queensland or Victoria. It has shown that the majority of low, mid and high-rise multi-storey buildings in Australia use RC walls as the major lateral load resisting system of the structure. This mostly consisted of (i) a system of isolated cast in-situ RC walls, (ii) a central or eccentric cast in-situ RC building core, or (iii) a combination of both former options. In recent years, the adoption of precast concrete walls and jointed precast building cores over traditional cast in-situ RC elements has become increasingly popular in low and mid-rise construction, particularly in the south-eastern Australian states.

RC walls and building cores are typically detailed with a continuous mat of vertical and horizontal reinforcement on each face of the wall, with uniform bar spacing across the length of the wall in both directions. While the detailing around openings and wall intersections slightly varied across the projects, the majority of buildings were detailed with additional ‘U’ bars matching the main reinforcement bar size and spacing at wall intersections, around openings and at the end regions of walls. Upper and lower bound values for various wall attributes from the case study buildings (e.g. aspect ratio, reinforcement ratio, axial load ratio etc.), have been summarised and presented in this Chapter. This data is presented as an aid for researchers when designing laboratory test specimens and numerical models of RC walls which accurately represent industry conditions.

Traditional force-based seismic design assumes strength is being traded for ductility through the use of the force reduction factor, which is a combination of a displacement ductility factor and structural performance (or overstrength) factor. Consideration needs to be given as to how the structure can develop the amount of ductility being assumed and what affect this will have on other structural elements in the building. Designers should adopt the ‘limited ductile’ structure classification to AS 1170.4 and AS 3600, which allows for displacement ductility and overstrength factors of 2.0 and 1.3 to be used respectively. The ‘moderately ductile’ classification should only be adopted when the much more onerous detailing requirements recommended within are adopted. The latter has a force reduction factor of 4.5, compared to the former which is equal to 2.6. The 73% higher force reduction factor does not come for free and requires the associated higher level of detailing to ensure the extra ductility is achieved.

Designers should adopt a newly proposed ‘non-ductile’ structure classification and assume a displacement ductility factor of 1.0 when detailing RC walls with low ductility reinforcement, central layers of vertical reinforcement or low percentages of vertical reinforcement (i.e. less than about 0.7%), which are all common detailing practices observed in precast walls.

The overall system behaviour of a building must be considered. Precast walls around the perimeter of a building (often referred to as load bearing only elements), cannot be assumed to take zero lateral load by assuming the central core takes 100% of the lateral load. While it may appear that these perimeter walls take minimal lateral load when distributing elastic forces based on initial or effective stiffnesses, the real forces could be somewhat different and larger when the overall system’s inelastic displacement is taken into consideration. When performing seismic analysis, which trades strength for ductility, displacement compatibility of the whole system needs to be thoroughly considered and established.
Chapter 4

Displacement Behaviour of Reinforced Concrete Walls

Significant amounts of research and experimental testing has been performed into the overall behaviour and seismic response of RC elements in the last 50 years. Much of this research however, was initially focused towards beams and columns, with RC wall research efforts only really being initiated in more recent times. This Chapter will firstly discuss the general displacement behaviour of RC walls and then more specifically, further investigate the behaviour with respect to limited ductile RC walls, which were identified in the previous chapter to be the dominant form of wall construction in Australia. Various failure mechanisms of RC walls are summarised and discussed within, including bar buckling and out-of-plane instability failures, which occur in the end regions of walls during reversed cyclic loading. Following this, a review of stress-strain material models for reinforcement and concrete are presented. Recommended mean material strengths for common grades of reinforcement and concrete specified in Australia are also proposed. The chapter is concluded with a review of RC wall testing that has been performed in literature. This review identified current gaps in RC wall testing and showed little test data is available for walls matching standard construction practices in Australia.

4.1 Displacement Behaviour of Reinforced Concrete

When undertaking displacement-based assessments of RC buildings, the force-displacement behaviour of RC elements and combined RC systems is essential. Traditionally the design of RC buildings involves assessing the ultimate limit state performance of individual elements under static or equivalent static lateral loads. The response is assumed to be linear and as such, the designer needs only consider the force behaviour (i.e. member moment or shear capacity).

The force behaviour of RC elements is well understood (e.g. [P1, P2, W5]) and codified (e.g. [X6, X8, X9, X10]), unlike the displacement behaviour, which in many respects, particularly in regards to the post-yield behaviour of limited ductile RC walls, is understood to a much lesser extent.
RC walls have various different types of deformation that need to be accounted for when assessing their total displacement. The two main types of deformation are flexure (i.e. Figure 4.1(a)) and shear (i.e. Figure 4.1(c)), which result from bending and shear stresses respectively. A portion of the flexure deformation, commonly referred to as the yield penetration displacement (i.e. Figure 4.1(d)), is often separated from the remaining flexural response of the wall because it deals with yield penetration and bond slip of the vertical reinforcement in the foundation element below. Similarly, when there is a large concentration of shear displacement in one location, it is often considered separately to the remaining shear deformations and referred to as sliding shear deformation (i.e. Figure 4.1(d)). Sliding shear deformation usually occurs in locations where there is a significant concentration of plastic tensile strain, such as in lightly reinforced walls that develop undesirable single crack plastic hinges (i.e. Figure 3.14(b)), or in areas of a wall where localised damage has occurred (i.e. bar buckling of the vertical reinforcement or crushing of the compression toe at the base of the wall). The latter was observed in the recent full-scale four-storey E-Defence shake-table testing performed in Japan [N1]. Furthermore, this behaviour is usually limited to lightly loaded walls, i.e. walls with low axial load ratios.

The last type of displacement that needs to be accounted for is caused from rigid body rotations (i.e. Figure 4.1(e)). This displacement is due to rotation of the foundation element supporting and normally results from rotational settlements in soft soils or other soil-structure interactions. The displacement due to rigid body rotation is simply equal to the base rotation multiplied by the height of the wall, i.e. $\theta_{base}H_w$.

![Figure 4.1: Deformation components of RC walls.](image)

The flexure (i.e. $\Delta_f$) and shear (i.e. $\Delta_s$) displacement of a cantilever wall element constructed from an isotropic material can be calculated using Equations 4.1 and 4.2 respectively. Using these Equations, a comparison of the total flexure versus shear displacement in a cantilever wall element can be developed (i.e. Figure 4.2). Figure 4.2(a) shows that for walls with an aspect ratio or height-to-depth ratio (i.e. $H_w/L_w$) greater than 1.0, flexure deformation dominates and accounts for more than 80% of the total displacement. Unfortunately, this comparison is fundamentally flawed with respect to an RC wall, as concrete is not an isotropic material. Furthermore, when concrete is subjected to bending and the resulting tensile stresses cause the extreme tensile fibre of the concrete to crack, the tensile stresses are taken solely by the reinforcement, resulting in a non-linear composite response of the element, further negating the assumptions that have been made to produce this flexure-shear relationship.
\[ \Delta_f = \frac{FH_w^3}{3EI} \] ... 4.1

\[ \Delta_s = \frac{FH_w}{GA_s} \] ... 4.2

Where:\n\( F \) = lateral load
\( H_w \) = wall height
\( E \) = modulus of elasticity (i.e. Young’s modulus)
\( I \) = moment of inertia of the wall
\( G \) = modulus of rigidity (i.e. shear modulus)
\( = \frac{E}{2(1 + \nu)} \)
\( \nu \) = Poisson’s ratio
\( A_s \) = shear area

\( \Delta_f \) and \( \Delta_s \) are the flexure and shear displacements, respectively.

\[ \frac{L_w}{t_w} \]

\( a \) rectangular wall
\( (L_w = 2 \text{ m}; t_w = 200 \text{ mm}) \)

\( b \) C section wall about strong axis
\( (L_w = 2 \text{ m}; b_w = 1.4 \text{ m}; t_w = 200 \text{ mm}) \)

**Figure 4.2:** Comparison of flexure and shear displacement.

What is useful about Figure 4.2, is that it shows how the relative contribution of flexure and shear deformations changes for a rectangular and non-rectangular C section wall. The C section wall has a much greater contribution of shear deformations for higher aspect ratio values. This is the case because the C section wall has approximately the same shear area (as the web of the C section wall is the same dimensions as the rectangular wall), yet a significantly larger moment of inertia.

The Eurocode [X10], Canadian [X32] and New Zealand [X8] standards for concrete structures generally considers all RC walls with aspect ratios less than 2.0 to be squat walls. Squat walls typically assume shear stresses and shear deformations to be the predominately response characteristics under lateral load. Walls with an aspect ratio greater than 2.0 could then generally be referred to as slender walls. This is a particularly binary assumption and it is likely that RC walls with aspect ratios from about 1.5 to 2.0 have closer to equal parts shear and flexure response. Paulay and Priestley [P1] propose that once the aspect ratio is greater than 4.0, the walls shear deformations become insignificant and can be neglected.
4.2 Lateral In-Plane Displacement Behaviour of Reinforced Concrete Columns

Significant research has been performed into the displacement capacity of ductile reinforced concrete columns (i.e. columns with high confinement reinforcement ratios) over the past two decades. It has only been in recent years that research efforts have been concentrated towards limited ductile columns, both locally and abroad (i.e. Lam et al. [L11]; Sezen and Moehle [S10, S11]; Elwood and Moehle [E1, E2]; Wilson, Lam and Rodsin [W13]; Wu et al. [W14]; Wibowo et al. [W7, W8, W15]; Ghannoum and Moehle [G7, G8]; and Wilson et al. [W9]).

4.2.1 Force-Displacement Models

Wilson et al. [W9] recommend a detailed and simplified force-displacement model for limited ductile RC columns for use in seismic analysis of structures to Australian building codes and standards. The detailed model, i.e. Figure 4.3(a), comprises five stages: cracking strength, yield strength, ultimate strength, lateral load failure and axial load failure. Whereas the simplified model, Figure 4.3(b), comprises of just two stages: yield strength and ultimate strength.

The simplified model was developed to be used as a ‘first tier’ analysis, which designer could use to quickly assess whether a structure was earthquake compliant using displacement principles. The simplified model is a very approximate and quite conservative approach that uses just the effective stiffness and design ultimate strength of a column and the relevant overstrength and ductility factors from the earthquake loading standard, AS 1170.4 [X4] for the level detailing specified in the design. The detailed model was proposed as a ‘second tier’ approach to be used when a more refined solution was required. The detailed model includes an empirical formula (Equation 4.3) for the drift capacity of limited ductile RC columns corresponding to complete structural collapse (i.e. axial load failure of the column). Wilson et al. [W9] Equation 4.3 using a comprehensive experimental testing program (i.e. [W6, W7, W8, W13, W15]).

![Figure 4.3: Wilson et al. [W9] force-displacement model for limited ductile RC columns.](image)

Using Equation 4.3 a series of charts were produced (as shown in Figure 4.4) to compare how three major parameters affect the maximum drift capacity, namely: axial load ratio, vertical reinforcement ratio and horizontal reinforcement ratio. Figure 4.4 shows that decreasing the axial load ratio, increasing the amount of confinement reinforcement and increasing the amount of vertical reinforcement all increase the maximum lateral drift capacity. The axial load ratio has the greatest impact on the lateral drift behaviour, whilst the vertical reinforcement has the least.
\[ \gamma_{af} = 5(1 + p_v)\left(\frac{1}{1-p_f}\right) + 7p_f + \frac{1}{5n} \]

Where:
- \( \gamma_{af} \) = lateral drift corresponding to axial load failure
- \( p_v \) = vertical reinforcement ratio (in %)
- \( p_v = \frac{A_{sv}}{(bD)} \) where: \( p_v \leq 2.0 \% \)
- \( p_f \) = confinement reinforcement ratio (in %)
- \( p_f = \frac{A_{sf}}{(bs)} \) where: \( p_f \leq 0.4 \% \)
- \( A_{sv} \) = total area of vertical reinforcement
- \( A_{sf} \) = total area of confinement reinforcement
- \( b \) = column width
- \( D \) = column depth
- \( s \) = confinement reinforcement spacing
- \( \beta = \frac{n}{n_b} \) where: \( \beta < 1.0 \)
- \( n = \frac{N^*}{(f'_c bD)} \) where: \( 0.1 \leq n < n_b \)
- \( n_b \) = axial load ratio corresponding to the balance point

**Figure 4.4:** Maximum drift capacity of 50 MPa 500x500 RC columns.
4.3 Lateral In-Plane Displacement Behaviour of Reinforced Concrete Walls

While considerable research and testing efforts have been performed to determine the displacement capacity of limited ductile RC columns, there has been considerably less attention towards limited ductile RC walls. Many research studies and experimental testing programs, both locally and internationally, have been performed generally with regards to RC walls, but the majority of these studies have been direct towards ductile RC walls.

4.3.1 Force-Displacement Models

Various researchers have proposed models for the displacement capacities at yield and ultimate for different geometry wall sections. However, little has been presented for the lateral drift corresponding to axial load failure (i.e. complete structural collapse), as summarised and shown in Table 4.1 and Figure 4.5. The Wibowo et al. [W16] and Wilson et al. [W9] models were developed primarily for limited ductile RC walls (i.e. unconfined walls). The FEMA 356 [X33] and ASCE/SEI 41 [X34] models can be used for both confined and unconfined walls (i.e. ductile and limited ductile respectively). The remaining models were essentially developed from wall testing in high seismic regions and as such, may not be appropriate for limited ductile wall response. Each model has been briefly discussed in the following sub-sections.

Table 4.1: Comparison of different force-displacement models for RC walls.

<table>
<thead>
<tr>
<th>Model</th>
<th>Rectangular walls</th>
<th>Other wall sections</th>
<th>$\Delta_{cr}$</th>
<th>$\Delta_y$</th>
<th>$\Delta_u$</th>
<th>$\Delta_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wibowo et al. [W16]</td>
<td>X</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Wilson et al. [W9]</td>
<td>X</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Panagiotakos and Fardis [P8]</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Su, Lam and Tsang [S12]</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Huang et al. [H5]</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FEMA 356 [X33] (flexure)</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FEMA 356 [X33] (shear)</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASCE/SEI 41 [X34] (flexure)</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASCE/SEI 41 [X34] (shear)</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Priestley and Kowalsky [P6, P9] *</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Adebar and Ibrahim [A2] *</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Sullivan et al. [S3] *</td>
<td></td>
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</tr>
<tr>
<td>Paulay [P10] *</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fibre element analysis†</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$\Delta_{cr}$ denotes cracking displacement.

$\Delta_y$ denotes yield displacement.

$\Delta_u$ denotes ultimate displacement, typically corresponds to the maximum lateral strength or the lateral strength dropping to 80% of the maximum.

$\Delta_c$ denotes collapse displacement.

* denotes curvature models that need to be converted to force-displacement by the designer, typically using an equivalent plastic hinge length, as discussed in Section 4.3.3.

† fibre element analysis models are discussed further in Section 4.3.2.
Figure 4.5: Force-displacement models for RC walls.
4.3.1.1 Wibowo, Wilson, Lam and Gad Force-Displacement Model

Wibowo et al. [W16] modified the force-displacement model developed for lightly reinforced limited ductile concrete columns ([W6, W7, W8]) to create a detailed model and a simplified model for limited ductile RC rectangular walls. The detailed model was developed for wall specimens being dominated by flexural actions and comprises four stages: cracking, yield, peak and ultimate displacements. The detailed model is shown in Figure 4.5(a).

4.3.1.2 Wilson, Wibowo, Lam and Gad Force-Displacement Model

Wilson et al. [W9] recommended a further simplified wall model (refer Figure 4.5(b)) to the simplified wall model presented by Wibowo et al. [W16]. This model comprises three stages: cracking, yielding and ultimate displacements and is similar to their simplified column model discussed previously (i.e. Figure 4.3(b)). The simplified wall model was developed with the intent of being used as a 'first-tier' analysis, which designers could use to quickly determine whether a structure was earthquake compliant using displacement principles. The model is quite conservative and somewhat imprecise, however it is very simple to use and requires just the cracking properties and design ultimate strength of the wall, an empirical effective stiffness and the overstrength and ductility factors from the earthquake loading standard, AS 1170.4.

4.3.1.3 Panagiotakos and Fardis Force-Displacement Model

Panagiotakos and Fardis [P8] performed a desktop study, utilising a database of over 1000 tests, to develop semi-empirical expressions for the lateral drift of RC members at yield and ultimate. The database included tests of various different RC members (beams, columns and walls) and testing regimes (cyclic and monotonic). The authors found their proposed relationships for both the yield drift and ultimate drift provided good average agreement with test results, but with large scatter, particularly for the yield drift. In this study, the ultimate drift is defined as the drift at which the resisting force of the wall has surpassed its maximum value and dropped to 80% of the maximum (i.e. 20% degradation in strength).

4.3.1.4 Su, Lam and Tsang Force-Displacement Model

Su et al. [S12] proposed a bi-linear force-displacement model for rectangular RC walls and columns. The yield drift in their model is calculated using an empirical equation developed using the results from experimental cyclic load tests of concrete members undertaken in Hong Kong. They propose the ultimate drift to be the lateral drift corresponding to the point at which 20% strength degradation occurs and cite the equation developed by Panagiotakos and Fardis [P8]. The proposed bilinear approximation can be seen in Figure 4.5(c).

4.3.1.5 Huang, Li, Lin and Hsu Force-Displacement Model

Huang et al. [H5] developed a process for calculating a bilinear force-displacement curve for RC members and is illustrated in Figure 4.5(d). The process was validated against test results by Chiou et al. [C7] and found to be in good agreement where the experimental results were within 5% and 10% of analytical values for displacement and force respectively. The model accounts for both flexure and shear deformations in their proposed equations for the notional yield and yield displacement. Further, in the paper guidance is given stating if the aspect ratio of the wall (i.e. $H_w/L_w$) is greater than 3 the shear deformation can be neglected and similarly, if the aspect ratio of the wall is less than 0.3 the flexure deformation can be neglected. The ultimate displacement is calculated using the ultimate curvature of the section and an equivalent plastic hinge length.
4.3.1.6 FEMA 356 Force-Displacement Model

FEMA 356 Prestandard and Commentary for the Seismic Rehabilitation of Buildings [X33] was prepared for the Federal Emergency Management Agency by the American Society of Civil Engineers. FEMA 356 provides guidelines for the seismic design and retrofit of buildings of various forms of construction, including: steel, concrete, masonry and timber. FEMA 356 provides generalised force-deformation models for RC walls controlled by either flexural behaviour or shear behaviour. FEMA 356 states if the aspect ratio (i.e. $H_w/L_w$) is greater than 3 the wall is considered to be slender and the model for flexural behaviour is to be adopted. If the aspect ratio is less than 1.5 the wall is considered to be squat and the model for shear behaviour is to be adopted. If the aspect ratio falls within these two limits the wall is considered to have a combination of both flexural and shear behaviour. The flexure and shear models are shown diagrammatically in Figures 4.5(e) and 4.5(f) respectively.

4.3.1.7 ASCE/SEI 41-06 Force-Displacement Model

ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings [X35] was published by the American Society of Civil Engineers in 2007 to assist practicing engineers and to further build upon the prestandard, FEMA 356, published in 2000. When ASCE/SEI 41-06 was published, the provisions for concrete structures were essentially the same as FEMA 356. A supplement to ASCE/SEI 41-06 was later published titled, ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings Supplement No. 1 [X34], which incorporated the latest research not captured in FEMA 356, resulting in a substantially more accurate assessment of the capacity of concrete components and structures. The flexure-controlled behaviour model for RC walls in ASCE/SEI 41-06 is the same as FEMA 356, which was just discussed above. Whereas the shear-controlled behaviour model for RC walls in ASCE/SEI 41-06 has changed considerably from the model originally presented in FEMA 356 and is shown diagrammatically in Figure 4.5(g).

4.3.1.8 Priestley and Kowalsky Moment-Curvature Model

Bilinear moment-curvature relationships for rectangular RC walls can be constructed using the yield and ultimate curvature formulas presented by Priestley and Kowalsky [P6, P9]. They found that the yield and ultimate curvatures were reasonably insensitive to axial load (for ranges: $0 \leq n \leq 0.15$) and reinforcement ratio and recommended Equations 4.4 and 4.5 respectively.

$$\phi_y = \frac{2.0\varepsilon_{sy}}{L_w} \pm 10\% \quad \ldots 4.4$$

$$\phi_u = \frac{0.072}{L_w} \pm 10\% \quad \ldots 4.5$$

4.3.1.9 Adebahr and Ibrahim Moment-Curvature Model

Adebar and Ibrahim [A2] proposed a trilinear moment-curvature model for non-rectangular RC members. They suggest that a simple bilinear moment-curvature model, such as the one defined above by Priestley and Kowalsky [P9], is not appropriate for non-rectangular sections and is only suited to rectangular members. The trilinear model features an upper and lower bound branch to represent the conditions of previously uncracked and severely cracked respectively (refer Figure 4.5(h)). They proposed an empirical formula to calculate the moment corresponding to the onset of each branch. Adebahr, Ibrahim and Bryson [A3] did a cyclic load test on a large-scale wall specimen and found good agreement with the trilinear model and the experimental test results.
4.3.1.10 Sullivan Priestley and Calvi Curvature Recommendations

Sullivan et al. [S3] proposed a series of formulae to calculate the notional yield curvature for different RC cross sections. The formulae have been summarised in Equations 4.6 to 4.12. Sullivan et al. [S3] note that the formulae for C and T cross sections have not been accurately validated and direct the designer to more appropriately undertake a fibre element analysis (refer Section 4.3.2) to determine yield curvature values for those sections.

\[
\phi_y = \frac{2.25 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{circular RC columns} \quad \ldots 4.6
\]

\[
\phi_y = \frac{2.10 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{rectangular RC columns} \quad \ldots 4.7
\]

\[
\phi_y = \frac{2.00 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{rectangular RC walls} \quad \ldots 4.8
\]

\[
\phi_y = \frac{1.40 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{C section RC walls for either bending about the strong axis or bending about the weak axis with the web in compression} \quad \ldots 4.9
\]

\[
\phi_y = \frac{1.80 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{C section RC walls for bending about the weak axis with the web in tension} \quad \ldots 4.10
\]

\[
\phi_y = \frac{1.75 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{T section RC walls with distribution reinforcement and the flange in compression} \quad \ldots 4.11
\]

\[
\phi_y = \frac{2.15 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{T section RC walls with distributed reinforcement and the flange in tension} \quad \ldots 4.12
\]

4.3.1.11 Paulay Curvature Recommendations

Paulay [P10] proposed a series of formulae to calculate the notional yield curvature for different RC cross sections. The formulae have been summarised in Equations 4.12 to 4.17.

\[
\phi_y = \frac{2.0 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{Rectangular RC walls with distributed reinforcement} \quad \ldots 4.13
\]

\[
\phi_y = \frac{1.8 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{Rectangular RC walls with concentrated reinforcement at each end} \quad \ldots 4.14
\]

\[
\phi_y = \frac{1.4 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{C section RC walls for bending about the strong axis} \quad \ldots 4.15
\]

\[
\phi_y = \frac{1.4 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{T section RC walls with the flange in compression} \quad \ldots 4.16
\]

\[
\phi_y = \frac{1.8 \varepsilon_{sy}}{L_w} \pm 10\% \quad \text{T section RC walls with the flange in tension} \quad \ldots 4.17
\]
4.3.1.12 Concluding Remarks

While there has been much research into the displacement behaviour of RC walls and various equations or models proposed, their still lacks a unified approach. Further, the majority of the models and formula discussed have been developed in conjunction with RC wall testing with ductile reinforcement detailing, as few experiment programs have been conducted which have limited ductile test specimens.

The majority of the models discussed have been developed based on experimental test results and as such, are only validated for certain types of failure mechanisms. Given that the failure mechanisms for limited ductile walls can differ from ductile walls (e.g. local bar buckling, which is discussed later in Section 4.3.4.3), directly translating these models across into Australian design practice could be either inappropriate or conservative.

A comprehensive force-displacement model for limited ductile RC walls verified from experimental limited ductile RC wall testing, is not available in the literature and represents a significant hurdle to implementing displacement-based design procedures in Australia or other regions of lower seismicity.

4.3.2 Fibre Element Analysis

The force-deformation behaviour of cracked reinforced concrete is generally considerably complex due to the inherent material non-linearity. Fibre-element analysis procedures, which utilise nonlinear stress-strain material relationships of both concrete and steel reinforcement, have been developed to confidently predict the moment-curvature behaviour of cracked RC members. While such procedures are capable of predicting accurate and reliable design results; care in selecting appropriate material stress-strain relationships is essential. The analysis procedure also takes into consideration axial pre-compression and vertical reinforcement of the cross section. Any confinement reinforcement present is accounted for in the concrete stress-strain model.


Fibre element analysis generally makes two fundamental assumptions in the analysis. The first being that plane sections remain plane and the second that the concrete below the neutral axis (i.e. the concrete in tension) is ignored once the cracking moment capacity of the section is exceeded. Both of these assumptions can be problematic: the first in low aspect ratio walls where significant diagonal shear cracking occurs, as this type of cracking can void the plane sections remain plane assumption; and the second as it ignores any tension stiffening behaviour of the cracked concrete.

Fibre element analysis is essentially a non-linear sectional analysis procedure that requires an iterative method to solve for the moment-curvature response of the section. The analysis process is described in the following steps:

1. Subdivide the cross section of the wall into horizontal segments of finite thickness along the depth of the section.
2. Select a reference strain for the extreme compressive fibre of the section.
3. Select an initial neutral axis depth, i.e. $d_n$. 

[92x579]
4. Calculate the strain for each discrete concrete compressive fibre and reinforcing bar using the initial reference strain selected in Step 2 and the neutral axis depth from Step 3.

5. Calculate the stress for each discrete concrete compressive fibre and reinforcing bar using non-linear stress-strain material models and the respective strain values from Step 4.

6. Calculate the force acting at each discrete concrete compressive fibre and reinforcing bar by multiplying the respective stress values from Step 5 and the area of the respective concrete compressive fibre or reinforcing bar.

7. Sum all the forces, i.e. concrete compression forces, reinforcement compression and tension forces and the pre-compression load acting on the section, together to check if equilibrium of the section is achieved. If equilibrium is not achieved, Steps 4 to 6 are repeated for different neutral axis depth values until equilibrium is achieved, i.e. the out-of-balance force is equal to zero.

8. Calculate the moment capacity of the section by 'summing the moments', which result from each internal force across the depth of the section, e.g. the compressive forces in the concrete slices, the compressive or tensile forces in the rebar or the pre-compression load on the wall.

9. Calculate the curvature of the section using the initial reference strain selected in Step 2 and the final neutral axis depth value that was determined for the section.

10. Record the moment capacity and curvature values determined for the section.

11. Repeat Steps 2 to 10 for different reference strain values such that a whole response range of moment-curvature values (as determined from Step 10) are calculated for the section.

12. Plot the moment-curvature response of the section using the values from Step 10.

This process has been illustrated in Figure 4.6.
4.3.3 Plastic Hinge Models

Using structural mechanics and beam theory the displacement of a cantilever RC wall could possibly be determined by double integrating the curvature distribution of the wall for the level of lateral load being considered. That is:

$$\Delta(z) = \int \phi(z) \, dz$$ \hspace{1cm} \ldots 4.18$$

However in practice this does not result in displacements that align with laboratory experiments. Priestley et al. [P4] notes that this approach ignores the tension shift effect, shear deformation, anchorage deformation and the negative stiffness that can occur post the maximum moment being reached (i.e. the moment capacity starts to decrease while the curvature continues to increase). To overcome this Priestley et al. [P4] proposed a concept of a plastic hinge region where the strain and curvature are taken to be constant and equal to the maximum value occurring at the base of wall and the curvature distribution up to yield is assumed to be linear, as illustrated in Figure 4.7.

The ultimate displacement of the wall is then taken as the sum of the yield displacement and plastic displacement (Equations 4.19 and 4.20). The plastic displacement is determined by multiplying the plastic curvature by the plastic hinge length and then by the height of the wall (note: the height of the wall is adjusted to allow for yield penetration in Equation 4.20).

$$\Delta_y = \frac{\phi_y (H_e + L_{sp})^2}{3}$$ \hspace{1cm} \ldots 4.19$$

$$\Delta_u = \Delta_y + \Delta_p$$
$$= \Delta_y + \theta_p \left[ H_e - \left( \frac{L_p}{2} - L_{sp} \right) \right]$$ \hspace{1cm} \ldots 4.20$$
$$= \Delta_y + (\phi_u - \phi_y) L_p \left[ H_e - \left( \frac{L_p}{2} - L_{sp} \right) \right]$$

Where:

$$L_p = kH_e + 0.1L_w + L_{sp}$$

$$k = 0.2 \left( \frac{f_{wu}}{f_{sy}} - 1 \right) \leq 0.08$$

$$L_{sp} = 0.022f_{sy}d_b$$

**Figure 4.7:** Idealisation of curvature distribution (reproduced from [P4]).
Tjhin, Aschheim and Wallace [T3] recommended a more straightforward empirical approach for converting bilinear moment-curvature curves to bilinear force-displacement curves. Their approach is expressed in Equations 4.21 and 4.22 and the coefficients in Table 4.2. Plastic hinge models have also been proposed by Paulay and Priestley [P1] and Panagiotakos and Fardis [P8], which are presented in a similar form to Priestley et al. [P4], i.e. function of $H_w, L_w, d_0$ and/or $f_{sy}$.

### Table 4.2: Yield displacement coefficient [T3].

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>$K_{\Delta}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.333</td>
</tr>
<tr>
<td>2</td>
<td>0.316</td>
</tr>
<tr>
<td>3</td>
<td>0.308</td>
</tr>
<tr>
<td>6</td>
<td>0.297</td>
</tr>
<tr>
<td>10</td>
<td>0.293</td>
</tr>
<tr>
<td>20</td>
<td>0.289</td>
</tr>
</tbody>
</table>

\[
\Delta_y = K_{\Delta}y_yH_e^2 \quad \ldots \quad 4.21
\]

\[
\Delta_u = \Delta_y + \Delta_p \\
= \Delta_y + \theta_p \left( H_e - \frac{L_p}{2} \right) \quad \ldots \quad 4.22
\]

\[
= \Delta_y + (\phi_u - \phi_y) L_p \left( H_e - \frac{L_p}{2} \right)
\]

Where: $L_p = \min(0.5L_w; h_{s,1})$

$h_{s,1} =$ storey height at the base of the wall

### 4.3.4 Failure Mechanisms

RC Walls subject to cyclic lateral load are susceptible to various different failure mechanisms, these include: flexure failure, i.e. compression failure of the concrete or tension fracturing of the vertical reinforcement; bar buckling of the vertical reinforcement; out-of-plane instabilities, which typically results in global out-of-plane buckling or premature concrete crushing failures; and shear failure. Each of these mechanisms will be discussed below.

#### 4.3.4.1 Flexure Failure

Flexure failure of a wall occurs when either the compression or tension capacity of the concrete or reinforcement respectively is exceeded in the section. Compression failure of the concrete usually occurs at relatively small compressive strain values, unless confinement reinforcement is provided in the wall, which can significantly increase the ultimate strain of the concrete. In contrast, tension failure of the reinforcement usually occurs after significant strain hardening of the reinforcement, allowing the development of large tensile strains and usually, a desirable ductile failure mechanism. However, ductile failure does not always occur, particularly for under reinforced sections or when low ductility reinforcement is adopted, as discussed in Sections 3.5.4 and 3.5.3 respectively.
A gradual softening and slow reduction in lateral strength generally occurs in RC walls where compression failure of the concrete governs. Whereas when tension failure of the reinforcement governs, there is usually a sudden drop in lateral strength, which is associated with tensile bars fracturing. This has been illustrated in Figures 4.8(a) and 4.8(b), which shows the moment-curvature response of two case study RC walls, which fail in flexure with compression face and tension-face controlled behaviour respectively.

(a) compression-face controlled wall

(b) tension-face controlled wall

Figure 4.8: Typical RC wall response failing in flexure – moment-curvature responses.

(a) compression-face controlled wall

(b) tension-face controlled wall

Figure 4.9: Typical response of an RC wall failing in flexure – extreme fibre strains.

(a) compression-face controlled wall

(b) tension-face controlled wall

Figure 4.10: Typical response of an RC wall failing in flexure – neutral axis depth ratios.
The maximum tensile strains in the extreme tensile reinforcing bar are much larger than the corresponding compressive strains in the tension-face controlled failure compared to the compression-face controlled failure (i.e. Figure 4.9(b) versus Figure 4.9(a)). In the compression-face controlled wall, the yield moment capacity (i.e. $M_y$), the point corresponding to the maximum concrete compressive strength being reached and the maximum strength of the wall (i.e. $M_{max}$) all occur in quick succession before the strength begins to deteriorate with increasing curvature. In the tension-face controlled wall, the point corresponding to the maximum concrete compressive strength being reached occurs at a relatively much larger curvature, after significant strain hardening of the reinforcement. After the maximum capacity of the wall is exceeded, there is a sudden drop in capacity, which corresponds to fracturing of the tensile reinforcement.

The behaviour could be surmised by saying the tension-face controlled wall allows for much ductility to develop without any strength deterioration occurring, whereas the compression-face controlled wall has a limited range of curvature to develop ductility across before undesirable amounts of softening and strength deterioration occurs. It should be noted that this statement is made with respect to limited ductile walls where no confinement reinforcement is provided. If confinement is provided and the concrete is able to sustain much larger compressive strains, different compression-face controlled behaviour would ensue to what has been described here.

Further, the neutral axis depth to overall depth ratio is typically smaller for walls failing in flexure with tension-face controlled behaviour. The neutral axis depth ratio is approximately 0.1 and 0.3 for the wall with tension face and compression-face controlled behaviour respectively, as shown in Figure 4.10.

### 4.3.4.2 Out-of-Plane Instabilities

Due to the cyclic response of structures that occurs during earthquake ground excitations, additional failure mechanisms in RC walls can be developed, which would otherwise not be seem in a monotonic response. This includes buckling of the vertical reinforcement and out-of-plane instabilities and buckling of the end regions of the wall.

An RC wall develops tension forces at one end of the wall and compression forces at the other when lateral load is applied to the wall. Under large lateral loads the vertical reinforcement in the end region of the wall yields and develops plastic tension strains to resist the applied moment. When the load decreases back to zero, the vertical reinforcement in the end region of the wall can still have significant residual plastic tension strains, resulting in horizontal cracks across the wall in this region not closing. When the lateral load is reversed, the end region which was just in tension, is now in compression and the residual plastic tension strains need to be eliminated via compression before the wall can close these cracks and regain its axial stiffness. This behaviour is illustrated in Figure 4.11.

When the end region of an RC wall goes from being in axial tension to reversed cyclic axial compression, the first failure mechanism that can occur is out-of-plane instability failure. This is typically in the form of global out-of-plane buckling of the end region of the wall, generally over the height of the plastic hinge or across the whole storey height of the wall. The global out-of-plane buckling occurs prior to the reversed cyclic compression load ‘closes’ the cracks that opened on the previous tension cycle. Prior to the cracks closing, the lateral stiffness of the wall is significantly reduced and, in this stage, it is easily susceptible to out-of-plane buckling. As the compression force on the wall increases and tries to overcome the previous plastic tension strains in the reinforcement, the wall can buckle sideways, where it then loses all axial strength. This out-of-plane buckling behaviour is illustrated in Figure 4.12.
Such wall failures were observed and documented following the recent 2010 Chile and 2011 Christchurch Earthquakes [E3, S6, W12]. Interestingly, this phenomenon was observed and presented nearly 25 years ago by Paulay and Priestley [P11], but little attention has been direct towards this area prior to these two earthquakes, with only one study identified by Chai and Elayer [C8]. Since these two earthquakes, many research efforts have been initiated, including two separate large scale testing programs by Dashti et al. [D1] and Rosso et al. [R1, R2, R3].
Paulay and Priestley [P11], using an experimental study comprising four ductile RC wall specimens and the fundamentals of reinforced concrete mechanics, developed a formula (Equation 4.23) to calculate the maximum tensile strain a boundary element of a wall can undergo prior to out-of-plane buckling occurring in the subsequent reversed load cycle. Chai and Elayer [C8] performed an experimental study consisting of 14 ductile RC columns meant to represent the boundary elements of ductile RC walls. Using the results of their experimental study they further refined the formula originally presented by Paulay and Priestley [P11] and proposed Equation 4.24. Both formulas are a function of the thickness to buckling length ratio of the wall. The buckling length of the wall is defined by Paulay and Priestley [P11] to be equal to the plastic hinge length, but no greater than 80% of the unrestrained floor-to-floor height of the wall.

\[
\varepsilon_{sm} = 8\beta \left(\frac{t_w}{L_o}\right)^2 \xi_c \quad \ldots 4.23
\]

\[
\varepsilon_{sm} = \frac{\pi^2}{2} \left(\frac{t_w}{L_o}\right)^2 \xi_c + 3\varepsilon_{sy} \quad \ldots 4.24
\]

Where:
- \( t_w \) = wall thickness
- \( L_o \) = buckling length
- \( \beta \) = the ratio of the distance to the outer layer of vertical reinforcement to the thickness of the wall
  \[ = \frac{d}{t_w} \]
- \( \xi_c \) = the critical normalised out-of-plane displacement
  \[ = 0.5(1 + 2.35m - \sqrt{5.53m^2 + 4.70m}) \]
- \( m \) = the mechanical reinforcement ratio of the boundary element
  \[ = \rho_v f_{sv} / f'_c \]

It was recently argued by Segura and Wallace [S13], using the results of an experimental testing program of ductile RC walls, that these failures observed in Chile and Christchurch were not from the ‘global’ out-of-plane buckling behaviour discussed above (i.e. Figure 4.12) but rather from ‘localised’ out-of-plane instabilities caused by asymmetric crushing/spalling of concrete and buckling of the vertical reinforcement.

### 4.3.4.3 Bar Buckling of the Vertical Reinforcement

The second failure mechanism that can occur due to reversed cyclic axial load of the end regions of RC walls is bar buckling of the vertical reinforcement. This mechanism is generally controlled by load reversal following large tensile strains rather than large compression demands. The behaviour that prevents bar buckling differs from ductile to limited ductile RC walls significantly. Ductile walls are usual detailed with significant amounts of transverse confinement reinforcement in the end regions of the walls where bar buckling would occur. This confinement allows the wall to develop compressive strains in the concrete significantly higher than what would otherwise to possible. The high compressive strains generally cause the unconfined cover concrete to spall, exposing the confined core and vertical reinforcement. The confinement reinforcement then serves a dual purpose of also restraining the vertical bars from buckling when in compression. Priestley et al. [P4] proposes Equation 4.25 for determining the minimum required spacing (i.e. s) of the transverse confinement reinforcement to allow the vertical reinforcement to achieve sufficient compressive strains without buckling.
Limited ductile RC walls, unlike ductile RC walls, are generally detailed with no transverse confinement reinforcement. Meaning there are no transverse confinement hoops present to prevent bar buckling from occurring. Instead, limited ductile walls rely purely on the cover concrete to prevent bar buckling of the vertical reinforcement. Spalling of the cover concrete in limited ductile walls would usually also be associated with local compressive failure of the wall, since there is generally no confinement reinforcement provided. The previous scenario for ductile walls where the cover concrete has spalled off leaving the exposed confined core and vertical reinforcement, is therefore not relevant for limited ductile walls. The bar buckling behaviour of vertical reinforcement in limited ductile walls has been further illustrated in Figure 4.13.

![Figure 4.13: Bar buckling behaviour of vertical reinforcement in limited ductile RC walls.](image)

4.3.4.4 Shear Failure

Shear failure is an undesirable failure mechanism and is usually associated with brittle fail modes. The shear strength of an RC wall is assumed to have a contribution from both the concrete and the transverse horizontal reinforcement. The shear reinforcement is generally the horizontal reinforcement, however in squat walls (e.g. $H_w/L_w \leq 1$), the shear reinforcement is usually taken as the minimum of the vertical and horizontal reinforcement [X6]. To ensure that a desirable flexural failure mode can develop, the wall must have sufficient horizontal shear reinforcement such that the shear capacity of the wall is greater than the equivalent horizontal force required to develop the flexural moment capacity at the base of the wall.

4.3.5 Strain Limits and Drifts Limits for Performance Based Design

Performance based seismic design is where a set of expected building performance objectives are defined at the beginning of a project and assigned associated seismic ground shaking intensities, for which this objective must be met. Appropriate material strain limits and inter-storey drift limits can then be assigned for each level to ensure the associated performance objective is achieved. Sullivan et al. [S3] proposed a set of material strain limits and inter-storey drifts for three performance objectives, being: no damage, repairable damage and no collapse. The strain and drift limits are summarised in Tables 4.3 and 4.4 respectively. It should be noted for limited ductile walls where no transverse confinement reinforcement is used, the ultimate compression strain of concrete using Equation 4.26 is simply 0.004, and therefore, 0.006 for no collapse.
Table 4.3: Sullivan et al. [S3] material strain limits.

<table>
<thead>
<tr>
<th>Material strain limit</th>
<th>No damage</th>
<th>Repairable damage</th>
<th>No collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strain, $\varepsilon_c$</td>
<td>0.004</td>
<td>$\varepsilon_{cu}$ (Equation 4.26)</td>
<td>1.5$\varepsilon_{cu}$</td>
</tr>
<tr>
<td>Reinforcement tension strain, $\varepsilon_s$</td>
<td>0.015</td>
<td>$0.6\varepsilon_{su} \leq 0.05$</td>
<td>$0.9\varepsilon_{su} \leq 0.08$</td>
</tr>
</tbody>
</table>

$$\varepsilon_{cu} = \min \left[ 0.004 + 1.4 \frac{P_0 f_{sy} f_{su} f_{c}}{f_{cc}'} ; 0.02 \right] \ldots 4.26$$

Table 4.4: Sullivan et al. [S3] material strain limits.

<table>
<thead>
<tr>
<th>Drift limit</th>
<th>No damage</th>
<th>Repairable damage</th>
<th>No collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildings with brittle non-structural elements</td>
<td>0.004</td>
<td>0.025</td>
<td>No limit</td>
</tr>
<tr>
<td>Buildings with ductile non-structural elements</td>
<td>0.007</td>
<td>0.025</td>
<td>No limit</td>
</tr>
<tr>
<td>Buildings with non-structural elements detailed to sustain building displacements</td>
<td>0.010</td>
<td>0.025</td>
<td>No limit</td>
</tr>
</tbody>
</table>

Little guidance is provided in the Australian Concrete code, AS 3600 [X6] for material strain limits and inter-storey drift limits. Some additional guidance is found in other supporting material (i.e. the commentary) and the earthquake loading code, AS 1170.4. The available guidance on strain and drift limits have been summarised in Tables 4.5 and 4.6 respectively. It’s worth noting that the ‘ultimate limit state’ performance objective translates to a no collapse objective, as discussed in Section 2.2.2.

Table 4.5: AS 3600 [X6] material strain limits.

<table>
<thead>
<tr>
<th>Material strain limit</th>
<th>Serviceability limit state</th>
<th>Ultimate limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strain, $\varepsilon_c$</td>
<td>NA*</td>
<td>0.003†</td>
</tr>
<tr>
<td>Reinforcement tension strain, $\varepsilon_s$</td>
<td>NA*</td>
<td>$\varepsilon_{su}$†</td>
</tr>
</tbody>
</table>

* AS 3600 makes no reference to material strain limits for the SLS lateral loading of structures.
† AS 3600 stipulates that the ultimate compressive strain limit of concrete is 0.003 when using the ‘rectangular stress block’ assumption. However, for advanced methods of analysis taking into account material stress-strain relationships AS 3600’s only requirement is that “the strain in the compression reinforcement does not exceed 0.003”.
‡ AS 3600 has no requirements for maximum tensile strain limits and hence it would be assumed that the code’s intent would be to take the ultimate value as the limit.
Table 4.6: AS 3600 [X6] inter-storey drift limits.

<table>
<thead>
<tr>
<th>Drift limit</th>
<th>Serviceability limit state</th>
<th>Ultimate limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>All structures</td>
<td>0.002*</td>
<td>0.015†</td>
</tr>
</tbody>
</table>

* AS 3600 does not explicitly mention or require inter-storey drift limits for the lateral loading of structures. AS 3600 Supp1 [X36] (i.e. the commentary) does however instruct designers to adopt a maximum inter-storey drift limit of 1/500 for SLS loading. This limit is intended for serviceability wind actions only.

† AS 3600 provides no guidance here, however the earthquake loading code, AS 1170.4 [X4] has an inter-storey drift limit of 1.5% for ultimate limit state loading.

4.3.6 Wall Properties Affecting Displacement Capacity

A parametric study using a fibre element analysis program was performed to better understand how various wall properties affect the displacement capacity and behaviour of RC walls. The fibre element analysis program was developed and coded by the author. The program firstly determined the moment-curvature response of the section using non-linear material models for concrete and reinforcement and then calculated a bilinear force-displacement response using an equivalent plastic hinge.

The AS 3600 Supp1 [X36] unconfined concrete stress-strain relationship and Priestley et al. [P4] reinforcement stress-strain relationship was adopted for the analysis. Both of these material models are discussed further in Section 4.4. Standard concrete grades (i.e. N40, N50 and S65) and reinforcement grades (i.e. D500N) available in Australia were selected. The mean material properties of these materials (which are discussed in Section 4.4.3) were used in the analysis.

The baseline section for the analysis was a 2,000 mm long and 200 mm thick wall with an effective height of 8,000 mm, resulting in a shear span ratio of 4.0. The axial load ratio on the wall was taken as 5%. N20-250 each face vertical reinforcement (i.e. grade D500N 20 mm nominal diameter reinforcing bars at 250 mm centres on each face of wall) and grade N50 concrete (i.e. concrete with a characteristic cylinder strength of 50 MPa) was assumed. The wall was assumed to have no transverse confinement reinforcement.

The moment-curvature response of the wall was converted to a bilinear response by projecting a line from the origin through the point of first yield up to the point corresponding to the yield curvature (i.e. \( \phi_y \)), which was taken as the point on this line corresponding to the ultimate moment capacity being reached. A straight horizontal line was then drawn from the yield curvature to the ultimate curvature (i.e. \( \phi_u \)). This process is illustrated in Figure 4.14.

The point of first yield was taken as the point on the response curve corresponding to yielding of the extreme tensile reinforcement or the maximum compressive strength of the concrete being reached in the extreme compressive fibre, whichever occurred first. The moment and curvature corresponding to the point of first yield has been defined as the yield moment capacity (i.e. \( M_y \)) and notional yield curvature (i.e. \( \phi_y \)) respectively. The ultimate moment capacity was taken to be the point when a concrete compressive strain of 0.004 or tensile reinforcement strain of 0.6\( \varepsilon_{eu} \) was reached. The tensile reinforcement strain limit of 0.6\( \varepsilon_{su} \) was selected with reference to [P4, S3] and does not necessarily capture or prevent out-of-plane instabilities or bar buckling failures.

The bilinear moment-curvature response was converted to a force-displacement response using the Priestley et al. [P4] plastic hinge model (i.e. Equations 4.19 and 4.20). The moment-curvature and bilinear force-displacement response of the baseline section is shown in Figure 4.15.
Different response parameters of the section were calculated as follows: the curvature ductility was taken as the ultimate curvature divided by the yield curvature, i.e. $\mu_\phi = \phi_u / \phi_y$; the displacement ductility was taken as the ultimate displacement divided by the yield displacement, i.e. $\mu_\Delta = \Delta_u / \Delta_y$; the overstrength factor was taken as the ultimate moment capacity divided by the yield moment capacity, i.e. $\Omega = M_u / M_y$; the force reduction factor was taken as the product of the overstrength factor and the displacement ductility, i.e. $R_f = \Omega \mu_\Delta$; the effective moment of inertia was taken as the slope of the initial branch of the bilinear moment-curvature response divided by the concrete modulus of elasticity, which was calculated in accordance with AS 3600.

The parametric study was performed by varying four parameters, however only two parameters were varied at one time. The first was the axial load ratio and the second was one of the remaining three parameters. The original baseline values were adopted for the other two parameters that remained constant. The parameters and the values used for each are summarised below (the original baseline values are shown in bold for the convenience of the reader):

1. Axial load ratio: 0%, 5%, 10%, 15% and 20%.
2. Wall length: 1,500, 2,000 and 2,500 mm.
3. Reinforcement ratio: N16-250 each face ($p_y = 0.009$), N20-250 each face ($p_y = 0.014$) and N24-250 each face ($p_y = 0.020$).
4. Concrete grade: N40 ($f'_c = 40$ MPa), N50 ($f'_c = 50$ MPa) and S65 ($f'_c = 65$ MPa).
Chapter 4: Displacement Behaviour of Reinforced Concrete Walls

The results of the parametric study are presented in Figures 4.16 and 4.17. The response parameters for the wall reported were: yield curvature, ultimate curvature, yield displacement, ultimate displacement, curvature ductility, displacement ductility, yield moment capacity, ultimate moment capacity, overstrength, force reduction factor, neutral depth to overall depth ratio and effective moment of inertia to gross moment of inertia ratio. Key observations from the results of the study have been summarised based on the four parameters that were varied.

**Axial load ratio:**
- The axial load ratio is the main parameter that controls the behaviour of the wall and affects all the response parameters considered except the yield curvature and yield displacement.
- Increasing the axial load ratio on the wall decreases the displacement capacity while simultaneously increasing the moment capacity, lateral strength and effective moment of inertia.
- Increasing the axial load ratio increased the depth of the neutral axis.
- Increasing the axial load ratio decreases the displacement ductility, overstrength and force reduction factor. The force reduction factor always decreased by a significant factor, typically between 2 to 3, when the axial load ratio increased from 0% to 20%.

**Wall length:**
- Wall length greatly affected the curvature and displacement of the wall, however it had little effect on the ductility, overstrength, force reduction factor, relative depth of the neutral axis or effective moment of inertia.
- The yield curvature and yield displacement were largely only affected by changes in the wall length, which is in line with observations by Priestley et al. [P4] and many others.

**Vertical reinforcement ratio:**
- Increasing the vertical reinforcement content had no effect on the yield curvature or displacement, however it did have some effect on the ultimate curvature and displacement for lower axial load ratios.
- Increasing the vertical reinforcement content had minimal effect on the force reduction factor for axial load ratios greater than about 5%.
- Increasing the vertical reinforcement content, increases the effective moment of inertia of the wall.
- The moment capacity of the wall increased proportionately with the reinforcement ratio, regardless of the wall’s axial load ratio.

**Concrete grade:**
- Increasing the concrete strength has little effect on the displacement ductility, overstrength and force reduction factor for axial load ratios greater than about 5%. Interestingly, when the axial load ratio was less than 5%, increasing the concrete strength, increased the displacement ductility of the section. This was because the increase in concrete strength allowed the depth to the neutral axis to decrease and hence increase the curvature of the section allowing larger tensile strains and stresses to be developed in the reinforcement.
- Increasing the concrete strength increased the moment capacity and lateral strength of the wall once the axial load ratio was greater than about 10%, however as the axial load ratio approached 0%, the concrete strength had little effect.
Figure 4.16: Parametric study into the behaviour of limited ductile RC walls (1 of 2).
Figure 4.17: Parametric study into the behaviour of limited ductile RC walls (2 of 2).
Material Models for Limited Ductile RC Walls

Non-linear analysis methods, such as the fibre element approach outlined in the previous section (i.e. Section 4.3.2), require accurate non-linear stress-strain models for both confined and unconfined concrete and reinforcement. The following two sub-sections, i.e. Sections 4.4.1 and 4.4.2, will present an overview of widely adopted stress-strain models for concrete and reinforcement respectively. This is followed by a third section, i.e. Section 4.4.3, which presents recommended mean material strengths for industry standard concrete and reinforcement grades in Australia.

4.4.1 Concrete

In the 1970s and 1980s researchers at the University of Canterbury began developing stress-strain models for confined and unconfined concrete. This initially resulted in the model proposed by Kent and Park [K2] in 1971, which was cited by a definitive and widely popular text of the time, Reinforced Concrete Structures by Park and Paulay [P2] published in 1975. Later in 1982, Park, Priestley and Gill [P12] and Scott, Park and Priestley [S14] developed and proposed a Modified Kent and Park model (refer Equations 4.27 to 4.30). The initial Kent and Park [K2] confined model only allowed for an increase in strain due to the confinement steel and assumed the confined maximum compressive stress was equal to the unconfined maximum compressive stress (i.e. in the original confined model \( K \) from Equation 4.29 was essentially equal to 1.0). Whereas, the modified version allowed for an increase in the maximum compressive stress based on the confinement steel.

Modified Kent and Park model:

\[
\sigma_c = K f'_c \left[ \frac{2 \varepsilon_c}{0.002K} - \left( \frac{\varepsilon_c}{0.002K} \right) \right] \quad \text{where: } \varepsilon_c \leq 0.002K \quad \ldots 4.27
\]

\[
\sigma_c = K f'_c \left[ 1 - Z(\varepsilon_c - 0.002K) \right] \geq 0.2K f'_c \quad \text{where: } \varepsilon_c > 0.002K \quad \ldots 4.28
\]

\[
K = 1 + \frac{\rho_s f_{sy,f}}{f'_c} \quad \ldots 4.29
\]

\[
Z = \frac{0.5}{3 + \frac{0.29f'_c}{145f'_c - 1000} + \frac{3}{4} \rho_s \sqrt{\frac{d_c}{s}} + 0.002K} \quad \ldots 4.30
\]

Where:

- \( \varepsilon_c \) = longitudinal strain in concrete
- \( \sigma_c \) = longitudinal stress in concrete
- \( f'_c \) = maximum unconfined compressive stress
- \( \rho_s \) = ratio of volume of hoop reinforcement to volume of concrete core
- \( f_{sy,f} \) = yield stress of confinement reinforcement
- \( d_c \) = depth/width of the confined core
- \( s \) = centre-to-centre spacing of confinement
The Modified Kent and Park model was then superseded in 1988 by the Mander, Priestley and Park [M23] model for confined and unconfined concrete (refer Equations 4.31 to 4.36 and Figure 4.18(a)). The Mander et al. [M23] model is a widely used and highly cited stress-strain material model for concrete. The widely popular text, Seismic Design of Reinforced Concrete and Masonry Buildings by Paulay and Priestley [P1] published in 1992 and the definitive text on displacement-based seismic design, Displacement-Based Seismic Design of Structures by Priestley et al. [P4] published in 2007, both cite and recommended the Mander et al. [M23] model. The Mander et al. [M23] model was developed using test specimens with circular, rectangular and square cross sections and concrete compressive strengths ranging from 25 to 41 MPa. This however, suggests the Mander et al. [M23] model could potentially not be appropriate for high-strength concrete.

**Mander, Priestley and Park Model:**

Unconfined cover concrete:

\[
\sigma_c = f'_c \left[ \frac{n \left( \frac{\varepsilon_c}{\varepsilon_{co}} \right)}{n - 1 + \left( \frac{\varepsilon_c}{\varepsilon_{co}} \right)^n} \right] \quad \text{where: } \varepsilon_c \leq 2 \varepsilon_{co} \quad \ldots 4.31
\]

\[
\sigma_c = \sigma'_c \left( 1 - \frac{\varepsilon_c - 2\varepsilon_{co}}{\varepsilon_{cs} - 2\varepsilon_{co}} \right) \quad \text{where: } 2\varepsilon_{co} \leq \varepsilon_c \leq \varepsilon_{cs} \quad \ldots 4.32
\]

Confined core concrete:

\[
\varepsilon_{cco} = \varepsilon_{co}[1 + 5(K - 1)] \quad \ldots 4.34
\]

\[
f'_cc = Kf'_c \quad \ldots 4.35
\]

Where:

- \( \varepsilon_c \) = longitudinal strain in concrete
- \( \sigma_c \) = longitudinal stress in concrete
- \( f'_c \) = maximum unconfined compressive stress
- \( f'_l \) = effective lateral confining stress (refer [M23])
- \( \varepsilon_{co} \) = 0.002
- \( \varepsilon_{cs} \) = spalling strain
- \( \varepsilon_{ccu} \) = ultimate strain of confined concrete
- \( \sigma'_c \) = unconfined concrete stress corresponding to \( 2\varepsilon_{co} \)
- \( n \) = \( E_c/(E_c - E_{sec}) \)
- \( E_c \) = 5000\( f'_c \)
- \( E_{sec} = f'_c/\varepsilon_{co} \) (unconfined) or \( f'_{cc}/\varepsilon_{cco} \) (confined)
Many years later in 2011 Karthik and Mander [K3] developed a new model for confined and unconfined concrete (refer Equations 4.37 to 4.39 and Figure 4.18(b)). The unconfined model was largely based on the unconfined model proposed previously by Collins, Mitchell and Macgregor [C9] in 1993, which is widely cited as an appropriate model for both normal and high strength unconfined concrete. The confined model was developed using the experimental results from Mander et al. [M23] and Li, Park and Tanaka [L13], which meant the range of compressive strengths it was validated against increased to 25 to 83 MPa. This would suggest that the latest Karthik and Mander [K3] confined and unconfined model should be appropriate for both normal and high strength concrete. Comparisons of the Mander et al. [M23] and Karthik and Mander [K3] models for unconfined and confined concrete are presented in Figures 4.19(a) and 4.19(b) respectively.

Karthik and Mander Model:

\[
\sigma_c = Kf'_c[1 - (1 - x)^n] \quad \text{where: } 0 \leq x < 1 \quad \ldots \text{4.37}
\]

\[
\sigma_c = Kf'_c - \left(\frac{Kf'_c - f_{cu}}{x_u - 1}\right)(x - 1) \quad \text{where: } 1 \leq x < x_u \quad \ldots \text{4.38}
\]

\[
\sigma_c = f_{cu}\left(\frac{x - x_f}{x_u - x_f}\right) \quad \text{where: } x_u \leq x \leq x_f \quad \ldots \text{4.39}
\]

Where:

<table>
<thead>
<tr>
<th>unconfined cover concrete</th>
<th>confined core concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K = 1 )</td>
<td>( K = \text{refer Equation 4.36} )</td>
</tr>
<tr>
<td>( x = \varepsilon_c/\varepsilon_{co} )</td>
<td>( x = \varepsilon_c/\varepsilon_{cco} )</td>
</tr>
<tr>
<td>( x_u = \varepsilon_{cu}/\varepsilon_{co} )</td>
<td>( x_u = \varepsilon_{cu}/\varepsilon_{cco} )</td>
</tr>
<tr>
<td>( x_f = \varepsilon_{sp}/\varepsilon_{co} )</td>
<td>( x_f = \varepsilon_{sf}/\varepsilon_{cco} )</td>
</tr>
<tr>
<td>( \varepsilon_{co} = 0.0015 + f'_c/70,000 )</td>
<td>( \varepsilon_{cco} = \varepsilon_{co}[1 + 5(K - 1)] )</td>
</tr>
<tr>
<td>( \varepsilon_{cu} = 0.0036 )</td>
<td>( \varepsilon_{ccu} = 5\varepsilon_{cco} )</td>
</tr>
<tr>
<td>( \varepsilon_{cs} = 0.012 - 0.0001f'_c )</td>
<td>( \varepsilon_{cf} = 0.004 + \varepsilon_{ccu} )</td>
</tr>
<tr>
<td>( f'_c = \text{maximum unconfined compressive stress} )</td>
<td>( f'_c = \text{maximum unconfined compressive stress} )</td>
</tr>
<tr>
<td>( f_{cu} = 12 \text{ MPa} )</td>
<td>( f_{ccu} = 12 + f'_c(K - 1) )</td>
</tr>
<tr>
<td>( n = E_c\varepsilon_{co}/f'_c )</td>
<td>( n = E_c\varepsilon_{cco}/(Kf'_c) )</td>
</tr>
<tr>
<td>( E_c = 5,000\sqrt{f'_c} )</td>
<td>( E_c = 5,000\sqrt{f'_c} )</td>
</tr>
</tbody>
</table>

The commentary to the Australian Standard for Concrete Structures, AS 3600 Supp1 [X36] cites the Thorenfeldt, Tomaszewicz and Jensen [T4] base curve for unconfined concrete, which is then calibrated based on various studies (refer Equation 4.40).

\[
\sigma_c = f'_c\left[\frac{n}{n - 1 + \left(\frac{\varepsilon_c}{\varepsilon_{cco}}\right)^n}\right] \quad \ldots \text{4.40}
\]
Where:

\[ \varepsilon_c = \text{longitudinal strain in concrete} \]
\[ \sigma_c = \text{longitudinal stress in concrete} \]
\[ f'_c = \text{maximum unconfined compressive stress} \]
\[ \varepsilon_{co} = 4.11 (f'_c)^{0.75} / E_c \]
\[ n = E_c / (E_c - E_{sec}) \]
\[ E_c = (\rho_c^{1.5}) \times \left(0.043 \sqrt{f'_c}\right) \quad \text{when } f'_c \leq 40 \text{ MPa} \]
\[ = (\rho_c^{1.5}) \times \left(0.024 \sqrt{f'_c} + 0.12\right) \quad \text{when } f'_c > 40 \text{ MPa} \]
\[ \rho_c = 2,400 \text{ kg/m}^3 \text{ for normal-weight concrete} \]
\[ E_{sec} = f'_c / \varepsilon_{co} \]
\[ k = 1 \quad \text{when } \varepsilon_c \leq \varepsilon_{co} \]
\[ = 0.67 + f'_c / 62 \geq 1.0 \quad \text{when } \varepsilon_c > \varepsilon_{co} \]

Figure 4.18: Stress-strain material models for confined and unconfined concrete.

Figure 4.19: Comparison of the Mander et al. [M23] and Karthik and Mander [K3] stress-strain material models for confined and unconfined concrete.
4.4.2 Reinforcement

The stress-strain behaviour of reinforcement is largely dependent on whether the reinforcement is produced in straight lengths or coils. The former generally has a very distinct yield plateau region before the onset of strain hardening occurs under monotonic loading. Whereas the latter generally has no distinguishable yield plateau region and the transition from elastic response to inelastic strain hardening occurs over a larger region of strain, as opposed to a very distinguished elastic to inelastic transition point. This varying stress-strain behaviour is shown below in Figure 4.20. Generally, 10 mm nominal diameter reinforcing bars are produced in coils, 12 mm nominal diameter reinforcing bars are produced in either coils or straight bars and 16 mm nominal diameter and greater reinforcing bars are produced as straight lengths.

![Stress-strain behaviour of reinforcement](image)

**Figure 4.20:** Typical stress-strain behaviour of reinforcement under monotonic loading.

The yield stress of bars produced in straight lengths is quite obvious upon inspecting a typical stress-strain response curve from a simple monotonic test. Bars produced in coils however, that have no clearly distinguished yield point, the yield stress is taken as the 0.2% proof stress [X29]. The 0.2% proof stress is taken as the stress corresponding to the intersection of the actual stress-strain response curve and a line drawn from 0.2% strain at a slope of $E_s$ (i.e. the elastic modulus of the bar).

The non-linear stress-strain behaviour of reinforcement can be modelled using a simple bilinear stress-strain model (i.e. Equations 4.41 and 4.42 and Figure 4.21). The bilinear model consists of two stages. The first stage is the elastic response stage that consists of a straight line from the origin with a slope of $E_s$ (i.e. the elastic modulus of the reinforcement) up until the yield stress of the bar is reached. The second is the inelastic response stage that consists of a second straight line from the yield point to the point corresponding to the ultimate strain (i.e. uniform elongation) and ultimate stress.

Priestley et al. [P4] proposed a more detailed approach for modelling the non-linear stress-strain behaviour of reinforcement that consisted of three stages (i.e. Equations 4.43 to 4.45 and Figure 4.21). The first stage is the elastic response stage and is the same as the bilinear model. The second is the yield plateau region that consists of a horizontal line (i.e. constant stress) from the yield point to the yield plateau point (i.e. $\epsilon_{yp}$). The third stage is the inelastic response stage that consists of a parabolic curve from the yield plateau point to the point corresponding to the ultimate strain and ultimate stress. The slope of the parabolic curve at the ultimate point is equal to zero.
Bilinear model:

\[ \sigma_s = E_s \varepsilon_s \quad \text{where: } \varepsilon_s \leq \varepsilon_{sy} \quad \ldots \text{4.41} \]

\[ \sigma_s = f_{sy} + (f_{su} - f_{sy}) \left( \frac{\varepsilon_s - \varepsilon_{sy}}{\varepsilon_{su} - \varepsilon_{sy}} \right) \quad \text{where: } \varepsilon_{sy} < \varepsilon_s \leq \varepsilon_{su} \quad \ldots \text{4.42} \]

Where:  
\( \varepsilon_s \) = longitudinal strain in the reinforcement  
\( \sigma_s \) = longitudinal stress in the reinforcement  
\( \varepsilon_{sy} \) = yield strain of the reinforcement (i.e. \( f_{sy}/E_s \))  
\( \varepsilon_{su} \) = ultimate strain of the reinforcement  
\( f_{sy} \) = yield stress of the reinforcement  
\( f_{su} \) = ultimate stress of the reinforcement  
\( E_s \) = modulus of elasticity of the reinforcement

Priestley, Calvi and Kowalsky model:

\[ \sigma_s = E_s \varepsilon_s \quad \text{where: } \varepsilon_s \leq \varepsilon_{sy} \quad \ldots \text{4.43} \]

\[ \sigma_s = f_{sy} \quad \text{where: } \varepsilon_{sy} < \varepsilon_s \leq \varepsilon_{sp} \quad \ldots \text{4.44} \]

\[ \sigma_s = f_{su} - (f_{su} - f_{sy}) \left( \frac{(\varepsilon_{su} - \varepsilon_s)^2}{(\varepsilon_{su} - \varepsilon_{sp})^2} \right) \quad \text{where: } \varepsilon_{sp} < \varepsilon_s \leq \varepsilon_{su} \quad \ldots \text{4.45} \]

Where:  
\( \varepsilon_s \) = longitudinal strain in the reinforcement  
\( \sigma_s \) = longitudinal stress in the reinforcement  
\( \varepsilon_{sy} \) = yield strain of the reinforcement (i.e. \( f_{sy}/E_s \))  
\( \varepsilon_{sp} \) = yield plateau strain of the reinforcement  
\( \varepsilon_{su} \) = ultimate strain of the reinforcement  
\( f_{sy} \) = yield stress of the reinforcement  
\( f_{su} \) = ultimate stress of the reinforcement  
\( E_s \) = modulus of elasticity of the reinforcement

Figure 4.21: Bilinear (left) and Priestley et al. [P4] (right) reinforcement stress-strain models.
Comparisons of the bilinear and Priestley et al. [P4] models are presented in Figures 4.22 and 4.23 respectively for rebar samples produced in a coil and as a straight length. These figures show that both the bilinear and Priestley et al. [P4] models predict the actual stress-strain behaviour of bars produced as straight lengths quite well. The bilinear model slightly over-predicts the stress at low strain values (i.e. between the yield point and the yield plateau point) and slightly under-predicts the stress at higher strain values (i.e. between the yield plateau point and the ultimate point). It appears as if both models under-predict the stress for bars produced in coils when the 0.2% proof stress is used to determine the yield stress. However, the Priestley et al. [P4] model with the yield plateau region set to zero predicts the stresses much better than the bilinear model. It is possible that the Priestley et al. [P4] model and the adoption of a higher 0.5% proof stress criteria for the yield stress may predict the response much better.

It was identified in the reconnaissance survey in Chapter 3 that the majority of RC walls are typically detailed using 16 mm or greater nominal diameter bars, which means that RC walls will usually always be constructed using bars that are produced in straight lengths. This means that either the bilinear model or Priestley et al. [P4] model would be appropriate for modelling the non-linear response of RC walls. Further, the bilinear model is generally being recommended given it’s a simpler model to operate and the wide variation of yield plateau strains that were identified in Section 4.4.3 (i.e. the higher certainly achieved using the Priestley et al. [P4] model is traded-off against the uncertainty regarding mean reinforcement material properties).

Figure 4.22: Bilinear reinforcement stress-strain model comparison to test data.

Figure 4.23: Priestley et al. [P4] reinforcement stress-strain model comparison to test data.
Reinforcement subject to cyclic loading, which in the previous reversed load cycle sustains inelastic plastic deformations, undergoes the bauchinger effect where the axial stiffness is slowly reduced as the yield point of the bar is approached. This behaviour results in no clearly distinguishable yield point in the reversed load cycle where the stiffness of the bar slowly degrades from the elastic modulus (i.e. $E_s$) to the inelastic modulus (i.e. $E_s'$) over a large region of strain, as shown in Figure 4.24. This behaviour is not dissimilar to the behaviour of reinforcement produced in coils discussed previously. The cyclic stress-strain response of reinforcement, taking into account the bauchinger effect, can be modelled using the Giuffre-Menegotto-Pinto [X37] model (i.e. Equations 4.46 to 4.50). It should also be noted that it is shown later in Section 5.4 that the back-bone response of cyclically loading reinforcement is approximately equal to the monotonic response if the bar does not buckle in the reversed compression load cycles.

\[
\varepsilon_s^* = \frac{\varepsilon_s - \varepsilon_{s,r,i-1}}{\varepsilon_{s,0,i} - \varepsilon_{s,r,i-1}} 
\]

\[
\sigma_s^* = B\varepsilon_s^* + \frac{(1 - B)\varepsilon_s^*}{[1 + (\varepsilon_s^*)^R_i]^{1/R_i}} 
\]

\[
R_i = R_0 - \frac{a_1\xi_i}{a_2 + \xi_i} 
\]

\[
\xi_i = \frac{\varepsilon_{s,r,i-1} - \varepsilon_{s,0,i-1}}{\varepsilon_{ SY}} 
\]

\[
\sigma_s = \sigma_{s}^* (\sigma_{s,0,i} - \sigma_{s,r,i-1}) + \sigma_{s,r,i-1} 
\]

Where: $B = \text{ratio of inelastic to elastic modulus, i.e. } E_s'/E_s$

$E_s' = \text{inelastic modulus, i.e. } (f_{su} - f_{sy})/(\varepsilon_{su} - \varepsilon_{sy})$

$R_0, a_1$ and $a_2$ are constants dependent on the reinforcing bar. In a study by Dhakal and Maekawa [D2] values of 20, 18.5 and 0.15 respectively were proposed.

\[\sigma_{s} = \frac{\sigma_{s}^* (\sigma_{s,0,i} - \sigma_{s,r,i-1}) + \sigma_{s,r,i-1}}{\varepsilon_{ SY}}\]

\[\sigma_{s}^* = B\varepsilon_s^* + \frac{(1 - B)\varepsilon_s^*}{[1 + (\varepsilon_s^*)^R_i]^{1/R_i}}\]

\[R_i = R_0 - \frac{a_1\xi_i}{a_2 + \xi_i}\]

\[\xi_i = \frac{\varepsilon_{s,r,i-1} - \varepsilon_{s,0,i-1}}{\varepsilon_{ SY}}\]

\[\sigma_s = \sigma_{s}^* (\sigma_{s,0,i} - \sigma_{s,r,i-1}) + \sigma_{s,r,i-1}\]

\[\text{Where: } B = \text{ratio of inelastic to elastic modulus, i.e. } E_s'/E_s\]

\[E_s' = \text{inelastic modulus, i.e. } (f_{su} - f_{sy})/(\varepsilon_{su} - \varepsilon_{sy})\]

\[R_0, a_1 \text{ and } a_2 \text{ are constants dependent on the reinforcing bar. In a study by Dhakal and Maekawa [D2] values of 20, 18.5 and 0.15 respectively were proposed.}\]

**Figure 4.24:** Giuffre-Menegotto-Pinto model for the cyclic response of reinforcement.
4.4.3  Expected Mean Material Strengths for Non-Linear Analysis

Non-linear analysis of RC elements and structures should be performed using expected mean material properties, as opposed to unrealistic codified characteristic values. AS 3600 requires all non-linear analysis methods to use "mean values of all relevant material properties" and AS 3600-Supp1 states that for "non-linear and other refined methods of analysis, actual stress-strain curves, using mean rather than characteristic values, should be used". While AS 3600 provides guidance to what the mean strength of concrete is, no guidance is provided as to what the mean properties of D500L and D500N reinforcement are (i.e. the standard reinforcement grades in Australia). The expected mean material strengths for standard grades of reinforcement and concrete used in Australia will be presented in the following two sub-sections, i.e. Sections 4.4.3.1 and 4.4.3.2 respectively.

4.4.3.1  Mean Reinforcement Properties

Reinforcement in Australia must comply with AS/NZS 4671 [X29], which allows reinforcement to be produced as one of three ductility grades: ‘L’ grade, i.e. low ductility class; ‘N’ grade, i.e. normal ductility class; or ‘E’ grade, i.e. seismic (earthquake) ductility class. L and N grade are primarily produced for Australia and E grade is primarily produced for New Zealand. E grade reinforcement is typically very difficult to source in Australia and as such, rarely specified in building or infrastructure projects. N grade reinforcement is usually specified as grade 500 deformed bars (i.e. D500N) and L grade reinforcement is usually specified as grade 500 deformed welded mesh (i.e. D500L). Grade 500 has a minimum characteristic yield stress of 500 MPa.

D500N bars are usually expressed by the nominal diameter of the bar prefixed by the letter N, e.g. N16 denotes a 16 mm nominal diameter bar that is grade D500N. D500L mesh is usually expressed by a 2-letter prefix, where the first letter denotes the mesh pattern (i.e. ‘S’ denotes square mesh and ‘R’ denotes rectangular mesh) and the second letter is an L to denote low ductility. This is then followed by a series of numbers that denote the bar diameter and spacing, e.g. SL92 denotes welded mesh with an 8.6 mm nominal diameter bar that is grade D500L and spaced at 200 mm in each direction.

The mean material properties for standard grades of reinforcement in Australia were determined using a database of tensile test results, which was provided by an independent materials testing laboratory that is contracted by industry suppliers to assess the code compliance of their reinforcing bars. The database included test results on bar samples from multiple industry suppliers over a period of 5 years from 2011 to 2015. The grades and bar sizes that comprise the database are summarised in Table 4.7 and Figure 4.25. The test results include values for the yield stress (i.e. \( f_{sy} \)), ultimate stress (i.e. \( f_{su} \)) and ultimate strain (i.e. \( \varepsilon_{su} \)). This database was also supplemented by tensile tests performed at Swinburne University of Technology between 2014 to 2017. These tests have also been summarised in Table 4.7. In additional to the properties mentioned above, the Swinburne tests in the database also included the yield plateau strain (i.e. \( \varepsilon_{sp} \)) of the bar.

Summaries of the mean reinforcement properties for each reinforcement grade are presented in Tables 4.9 and 4.11 for the independent testing laboratory and Swinburne test data respectively. The coefficients of variation for each reinforcement property presented in these tables are summarised in Tables 4.10 and 4.12 respectively. Histogram and theoretical normal distributions of the independent testing laboratory tensile test data for the yield stress, strain hardening ratio and ultimate strain are presented in Figure 4.26.
Table 4.7: Summary of bar sizes and grades in the reinforcement database.

<table>
<thead>
<tr>
<th>Tests performed by</th>
<th>Grade</th>
<th>Bar sizes tested</th>
<th>Number of samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Independent testing laboratory</td>
<td>D500L</td>
<td>SL82, SL92 and SL102</td>
<td>2128</td>
</tr>
<tr>
<td>Independent testing laboratory</td>
<td>D500N</td>
<td>N10, N12, N16, N20, N24, N28 and N32</td>
<td>3979</td>
</tr>
<tr>
<td>Independent testing laboratory</td>
<td>D500E</td>
<td>E12, E16, E25 and E32</td>
<td>150</td>
</tr>
<tr>
<td>Swinburne University of Technology</td>
<td>D500L</td>
<td>L11.9</td>
<td>5</td>
</tr>
<tr>
<td>Swinburne University of Technology</td>
<td>D500N</td>
<td>N10, N12, N16, N20, N24 and N28</td>
<td>260</td>
</tr>
<tr>
<td>Swinburne University of Technology</td>
<td>D300E</td>
<td>E12</td>
<td>5</td>
</tr>
<tr>
<td>Swinburne University of Technology</td>
<td>D500E</td>
<td>E12 and E16</td>
<td>15</td>
</tr>
</tbody>
</table>

Figure 4.25: Summary of bar sizes tested for each grade in the reinforcement database (independent materials testing laboratory data only).

Mean reinforcement properties for standard grades of reinforcement (i.e. D500L, D500N and D500E) were determined using the database of reinforcement tensile test results and are summarised in Table 4.8 below. These values can be used to for constructing bilinear stress-strain curves (i.e. Figure 4.27), which can then be adopted for undertaking non-linear analysis methods for RC structures.

Table 4.8: Recommended mean reinforcement properties for non-linear analysis.

<table>
<thead>
<tr>
<th>Grade</th>
<th>$f_{sy}$</th>
<th>$f_{su}$</th>
<th>$\varepsilon_{su}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>D500L</td>
<td>585</td>
<td>620</td>
<td>3.3%</td>
</tr>
<tr>
<td>D500N</td>
<td>550</td>
<td>660</td>
<td>9.5%</td>
</tr>
<tr>
<td>D500E</td>
<td>530</td>
<td>660</td>
<td>13%</td>
</tr>
</tbody>
</table>
yield stress (MPa), i.e. $f_{xy}$

strain hardening ratio, i.e. $f_{su}/f_{xy}$

ultimate strain (%), i.e. $\epsilon_{su}$

**Figure 4.26:** Histogram and theoretical normal distributions of reinforcement test results (independent materials testing laboratory data only).
**Figure 4.27:** Recommended mean reinforcement stress-strain curves for non-linear analysis.

**Table 4.9:** Mean reinforcement properties (independent laboratory results).

<table>
<thead>
<tr>
<th>Grade</th>
<th>$f_{sy}$</th>
<th>$f_{su}$</th>
<th>$f_{su}/f_{sy}$</th>
<th>$\varepsilon_{sp}$</th>
<th>$\varepsilon_{su}$</th>
<th>Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>D500L</td>
<td>587 MPa</td>
<td>619 MPa</td>
<td>1.06</td>
<td>–</td>
<td>3.3%</td>
<td>2128</td>
</tr>
<tr>
<td>D500N</td>
<td>551 MPa</td>
<td>660 MPa</td>
<td>1.20</td>
<td>–</td>
<td>9.5%</td>
<td>3979</td>
</tr>
<tr>
<td>D500E</td>
<td>531 MPa</td>
<td>661 MPa</td>
<td>1.24</td>
<td>–</td>
<td>13.2%</td>
<td>150</td>
</tr>
</tbody>
</table>

**Table 4.10:** Coefficient of variation for mean reinforcement values (independent laboratory).

<table>
<thead>
<tr>
<th>Grade</th>
<th>$f_{sy}$</th>
<th>$f_{su}$</th>
<th>$f_{su}/f_{sy}$</th>
<th>$\varepsilon_{sp}$</th>
<th>$\varepsilon_{su}$</th>
<th>Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>D500L</td>
<td>0.053</td>
<td>0.050</td>
<td>0.024</td>
<td>–</td>
<td>0.254</td>
<td>2128</td>
</tr>
<tr>
<td>D500N</td>
<td>0.053</td>
<td>0.057</td>
<td>0.063</td>
<td>–</td>
<td>0.307</td>
<td>3979</td>
</tr>
<tr>
<td>D500E</td>
<td>0.046</td>
<td>0.045</td>
<td>0.036</td>
<td>–</td>
<td>0.106</td>
<td>150</td>
</tr>
</tbody>
</table>

Note: the standard deviation can be calculated by multiplying these coefficient of variation values by the respective mean strength value in Table 4.9.

**Table 4.11:** Mean reinforcement properties (Swinburne).

<table>
<thead>
<tr>
<th>Grade</th>
<th>$f_{sy}$</th>
<th>$f_{su}$</th>
<th>$f_{su}/f_{sy}$</th>
<th>$\varepsilon_{sp}$</th>
<th>$\varepsilon_{su}$</th>
<th>Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>D500L</td>
<td>577 MPa</td>
<td>626 MPa</td>
<td>1.09</td>
<td>–</td>
<td>3.2%</td>
<td>5</td>
</tr>
<tr>
<td>D500N</td>
<td>571 MPa</td>
<td>669 MPa</td>
<td>1.17</td>
<td>2.4%</td>
<td>10.9%</td>
<td>260</td>
</tr>
<tr>
<td>D300E</td>
<td>304 MPa</td>
<td>450 MPa</td>
<td>1.48</td>
<td>2.2%</td>
<td>18.9%</td>
<td>5</td>
</tr>
<tr>
<td>D500E</td>
<td>556 MPa</td>
<td>688 MPa</td>
<td>1.24</td>
<td>1.9%</td>
<td>12.4%</td>
<td>15</td>
</tr>
</tbody>
</table>

**Table 4.12:** Coefficient of variation for mean reinforcement values (Swinburne).

<table>
<thead>
<tr>
<th>Grade</th>
<th>$f_{sy}$</th>
<th>$f_{su}$</th>
<th>$f_{su}/f_{sy}$</th>
<th>$\varepsilon_{sp}$</th>
<th>$\varepsilon_{su}$</th>
<th>Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>D500L</td>
<td>0.010</td>
<td>0.005</td>
<td>0.007</td>
<td>–</td>
<td>0.040</td>
<td>5</td>
</tr>
<tr>
<td>D500N</td>
<td>0.064</td>
<td>0.060</td>
<td>0.037</td>
<td>0.381</td>
<td>0.104</td>
<td>260</td>
</tr>
<tr>
<td>D300E</td>
<td>0.012</td>
<td>0.002</td>
<td>0.011</td>
<td>0.104</td>
<td>0.011</td>
<td>5</td>
</tr>
<tr>
<td>D500E</td>
<td>0.030</td>
<td>0.039</td>
<td>0.012</td>
<td>0.177</td>
<td>0.068</td>
<td>15</td>
</tr>
</tbody>
</table>

Note: the standard deviation can be calculated by multiplying these coefficient of variation values by the respective mean strength value in Table 4.11.
It was observed while analysing the database that the bar size did seem to affect the mean properties (i.e. \(f_{sy, L}\), \(f_{sy, U}\) or \(\varepsilon_{su}\)), with no bar size-based trend being apparent. The yield plateau strain of D500N reinforcement was estimated using the Swinburne database to be about 2.4%. However, the coefficient of variation was considerable high at 0.38 (refer Table 4.12). Particularly in reference to the coefficient of variation for the other parameters for the same dataset, which ranged from 0.04 to 0.10 (refer Table 4.12).

A statistical analysis was performed on the database of test results to assess compliance of each respective grade of reinforcement against AS/NZS 4671. The analysis was only performed on the independent materials testing laboratory data, since that dataset included samples from five different suppliers performed consistently over a five-year period. Meaning the database likely contains test samples from over a thousand different manufacturing runs, making it a very comprehensive database of test results. Whereas the Swinburne test data, while still consisting of quite a number of test samples (285 total), does not consist of test samples from nearly as many manufacturing runs.

AS/NZS 4671 has lower and upper bound characteristic value limits for the yield stress, strain hardening ratio and ultimate strain. The standard specifies 5 and 95 percentile limits for the lower and upper characteristic values respectively for the yield stress, however for the strain hardening ratio and ultimate strain these values are widened to 10 and 90 percentile limits. Although it should be noted that the upper characteristic limit for the strain hardening ratio only applies to grade D500E reinforcement and grades D500L and D500N have no upper characteristic limit. Also, all grades of reinforcement have no upper characteristic limit for the ultimate strain.

The relevant lower and upper characteristic limits were calculated for each grade of reinforcement and are summarised and compared against the minimum and maximum values specified in AS/NZS 4671 in Table 4.13. The theoretical normal distributions and characteristic values for the yield stress, strain hardening ratio and ultimate strain are presented in Figures 4.28, 4.29 and 4.30 for grades D500L, D500N and D500E respectively. The statistical analysis showed that all the characteristic limits were met for the grade D500N reinforcement, however for the grades D500L and D500E reinforcement, the lower characteristic strain hardening ratio and lower characteristic yield stress requirement respectively were not achieved (refer Table 4.13).

<table>
<thead>
<tr>
<th>Property</th>
<th>Grade D500L Actual</th>
<th>Grade D500L Limit</th>
<th>Grade D500N Actual</th>
<th>Grade D500N Limit</th>
<th>Grade D500E Actual</th>
<th>Grade D500E Limit</th>
<th>Characteristic requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>(f_{sy, L}) (MPa)</td>
<td>536 (\geq) 500</td>
<td>(\leq) 500</td>
<td>503 (\geq) 500</td>
<td>(\leq) 500</td>
<td>(\text{491} \geq) 500</td>
<td>(\leq) 500</td>
<td>(C_{vl}: p = 0.95)</td>
</tr>
<tr>
<td>(f_{sy, U}) (MPa)</td>
<td>637 (\leq) 750</td>
<td>(\geq) 750</td>
<td>598 (\leq) 650</td>
<td>(\geq) 650</td>
<td>571 (\leq) 600</td>
<td>(\geq) 600</td>
<td>(C_{vu}: p = 0.05)</td>
</tr>
<tr>
<td>([f_{su}/f_{sy}]_{L})</td>
<td>1.02 (\geq) 1.03</td>
<td>(\leq) 1.03</td>
<td>1.10 (\geq) 1.08</td>
<td>(\leq) 1.08</td>
<td>1.19 (\geq) 1.15</td>
<td>(\leq) 1.15</td>
<td>(C_{vl}: p = 0.90)</td>
</tr>
<tr>
<td>([f_{su}/f_{sy}]_{U})</td>
<td>(\text{no requirement})</td>
<td>(\text{no requirement})</td>
<td>(\text{1.30} \leq) 1.40</td>
<td>(\geq) 1.40</td>
<td>(\text{1.40} \leq) 1.60</td>
<td>(\geq) 1.40</td>
<td>(C_{vu}: p = 0.10)</td>
</tr>
<tr>
<td>(\varepsilon_{su, L}) (%)</td>
<td>2.2 (\geq) 1.5</td>
<td>(\leq) 1.5</td>
<td>5.7 (\geq) 5.0</td>
<td>(\leq) 5.0</td>
<td>11.4 (\geq) 10.0</td>
<td>(\leq) 10.0</td>
<td>(C_{vl}: p = 0.90)</td>
</tr>
</tbody>
</table>
The lower characteristic yield stress for the D500E reinforcement was 491 MPa, which is 1.8% lower than the minimum requirement of 500 MPa. Given the relatively small database of samples for grade D500E (150 compared to 3979 for grade D500N, refer Table 4.7) and the fact it is under by only 1.8%, non-code compliance in this instance is not concerning. The lower characteristic strain hardening ratio of 1.02 for the grade D500L reinforcement, is however somewhat concerning, as this value is (relatively) somewhat lower than the characteristic value of 1.03.

In recent years it has been anecdotally suggested that some overseas manufacturers who sell both D500N and D500E reinforcement only produce D500E reinforcement and supply D500E reinforcement 'badged' as D500N to the Australian market. The histogram plots in Figures 4.26(b) and 4.26(c) suggest this may in fact be the case, since the distribution of results for the D500N samples clearly looks like two overlapping normal distribution curves, suggested two different and dominant manufacturing targets were used for the D500N reinforcement.

The database of results from the independent materials testing laboratory did not generally have the results separated by suppliers. However, a small subset of N16 bars, that consisted of 175 samples from five different suppliers was separated by the bar supplier.
This reduced subset of data was used to assess whether different manufacturers had considerably different manufacturing targets. It was observed that four of the five suppliers appeared to have very similar manufacturing targets, while the fifth supplier had a considerably different manufacturing target. This can clearly be observed in Figures 4.31(b) and 4.31(c), which show histogram plots and theoretical normal distributions of the strain hardening ratio and the ultimate strain respectively. In these figures the samples are separated into two groups: suppliers 1–4 and supplier 5. Figure 4.31 suggests that supplier 5 is clearly manufacturing reinforcement as grade D500N and suppliers 1–4 could possibly be manufacturing reinforcement as grade D500E and then supplying it to the Australian marketplace as D500N.

Figure 4.31: Supplier analysis of N16 bars, grade D500N.

4.4.3.2 Mean Concrete Properties

Concrete is usually specified for a given characteristic 28-day compressive strength (i.e. $f'_c$). Concrete in Australia is normally specified as a ‘normal’ grade, which for walls would generally be either N32, N40 or N50. These grades have characteristic 28-day compressive strengths of 32, 40 and 50 MPa respectively. Alternatively, ‘special’ grades can be specified, which are essentially high-strength grades and generally consist of S65, S80 and S100. These grades have characteristic 28-day compressive strengths of 65, 80 and 100 MPa respectively. Normal grades are usually specified by the 28-day compressive strength only, whereas the special grades (i.e. the high-strength grades) are usually specified by the 28-day compressive strength and additional information such as the workability, water/binder ratios, types and quantities of admixtures, water-reducing agents, aggregate types, shrinkage requirements, etc. [X36].
For non-linear analysis methods however, the mean in-situ strength of concrete (i.e. $f_{cmi}$) should be used, not the characteristic strength. The mean in-situ strength of concrete can be calculated using Equation 4.51.

$$f_{cmi} = K_1 K_2 K_3 f'_c$$  \(\ldots 4.51\)

Where: $K_1$ accounts for the difference between the characteristic 28-day cylinder strength and the mean 28-day cylinder strength, i.e. $K_1 = f_{cml}/f'_c$; $K_2$ accounts for the difference between the mean cylinder strength and the actual in-situ strength (these values differ since the cylinders are usually cured in a 28-day water bath and the structure obviously is not), i.e. $K_2 = f_{cmi}/f_{cm}$; and $K_3$ accounts for the difference between the 28-day strength of the concrete and the long-term strength of the concrete (i.e. the strength gain with time).

The current version of the Australian concrete code, AS 3600 [X6] proposes values for the mean strength of concrete using Equation 4.52, which is proposed in the code's commentary, AS 3600 Supp1 [X36]. AS 3600 then proposes that the difference between the in-situ strength and the mean strength is 0.9, i.e. $K_2 = 0.9$.

$$f_{cm} = (1.2875 - 0.001875 f'_c) f'_c$$  \(\ldots 4.52\)

A recent study by Foster et al. [F1] included a thorough assessment of what the actual mean strength is for standard concrete grades in Australia. This study used a database of 28-day test reports from the Cement Concrete and Aggregates Association (CCAA), which included over 30,000 cylinder tests performed in 12 locations across Australia (i.e. Sydney, Melbourne, Brisbane, Perth, Adelaide, Hobart, Northern Territory, NSW regional, VIC regional, QLD regions, WA regional and SA regional). The mean strength values proposed are summarised in Table 4.14. Foster et al. [F1] also proposed a mean to in-situ reduction factor of 0.88, i.e. $K_2 = 0.88$.

Table 4.14: Mean strength values for standard concrete grades in Australia [F1].

<table>
<thead>
<tr>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_c$ (MPa)</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>N20</td>
</tr>
<tr>
<td>N25</td>
</tr>
<tr>
<td>N32</td>
</tr>
<tr>
<td>N40</td>
</tr>
<tr>
<td>N50</td>
</tr>
<tr>
<td>S65</td>
</tr>
<tr>
<td>S80</td>
</tr>
</tbody>
</table>

The difference between the long-term strength of concrete and the 28-day strength can be difficult to assess since under the correct conditions, the strength of concrete can theoretical increase indefinitely at a logarithmic rate with respect to time. It is commonly thought that this is the case for concrete generally, however for in-situ RC structures this is not necessarily the case. In-situ concrete typically does not continue to strengthen with age, which is in contrast to 'continuously' moist cylinder samples, which can seem to continue to gain strength almost indefinitely.
A thorough discussion on the long-term strength development of concrete is provided in Neville [N2], including the overview of a studying looking at the strength development of well cured cylinder samples and core samples from the same batch of concrete over a one-year period. This study showed no long-term strength gain of the in-situ concrete after 28-days (refer Figure 4.32). In-lieu of this, the long-term in-situ mean strength of concrete is being proposed to be equal to the 28-day in-situ mean strength, i.e. $K_3 = 1.0$.

![Figure 4.32: Strength development of concrete cores made with type 1 (i.e. GP) cement (redrawn from [N2]).](image)

Concrete cylinder compressive strength data was also obtained from the independent materials testing laboratory. The maximum compressive strength test data received for concrete though, was significantly less than the amount of tensile test data received for reinforcement. This data included 28-day cylinder compressive strength tests on N20, N32, N40 and S100 grades of concrete, where each respective grade was from the same supplier. A statistical analysis for each grade of concrete was performed and the results are summarised in Table 4.15. Histogram and theoretical normal distributions for the N32 and N40 concrete grades are presented in Figures 4.33(a) and 4.33(b) respectively. The actual lower 5 percentile characteristic compressive strengths were calculated for each grade and were observed to be generally lower than the specified characteristic strengths, except in the case of the high-strength 100 MPa mix where is was slightly higher (Table 4.15).

<table>
<thead>
<tr>
<th>Grade</th>
<th>$[f'_c]_s$ (MPa)</th>
<th>$f_{cm}$ (MPa)</th>
<th>$f_{cm} / [f'_c]_s$</th>
<th>$\sigma$ (MPa)</th>
<th>COV</th>
<th># of samples</th>
<th>$[f'_c]_A$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N20</td>
<td>20</td>
<td>23.1</td>
<td>1.153</td>
<td>2.3</td>
<td>0.099</td>
<td>8</td>
<td>19.2</td>
</tr>
<tr>
<td>N32</td>
<td>32</td>
<td>38.5</td>
<td>1.203</td>
<td>4.1</td>
<td>0.129</td>
<td>18</td>
<td>31.7</td>
</tr>
<tr>
<td>N40</td>
<td>40</td>
<td>44.8</td>
<td>1.119</td>
<td>4.6</td>
<td>0.103</td>
<td>12</td>
<td>37.2</td>
</tr>
<tr>
<td>S100</td>
<td>100</td>
<td>113.5</td>
<td>1.135</td>
<td>7.8</td>
<td>0.069</td>
<td>6</td>
<td>100.6</td>
</tr>
</tbody>
</table>

$[f'_c]_s$ denotes the specified lower 5% characteristic compressive strength.

$[f'_c]_A$ denotes the actual lower 5% characteristic compressive strength calculated from the test data.
Figure 4.33: Statistical analysis of concrete cylinder compressive strength test data.

The mean strength to characteristic strength ratios calculated using this data were generally lower than the Foster et al. [F1] values summarised in Table 4.14, as shown in Figure 4.34. Further, good correlation between the AS 3600 recommendations, the Foster et al. [F1] data and the independent material testing laboratory data was generally not observed.

There are many factors that affect the compressive strength of concrete, which results in different suppliers having very different mix designs and objectives (e.g. mean strengths and standard deviations) for the same strength grade. This results in there being much more uncertainty when trying to predict the mean properties compared to reinforcement. This is evident by the much higher ratio of mean to characteristic compressive strengths and coefficient of variations. For common concrete grades typically specified in RC walls (i.e. N40, N50 and S65 – refer Figure 3.12) the mean to characteristic compressive strength ratio ranged from 1.21 to 1.23 and the coefficient of variation ranged from 0.091 to 0.097 (Table 4.14). Whereas, for D500N reinforcement, the mean to characteristic yield stress ratio and the coefficient of variation was significantly lower at 1.10 and 0.053 respectively (Tables 4.9 and 4.10). Due the high uncertainty in predicting a mean compressive strength, the author recommends simply adopting a mean to characteristic strength ratio of 1.2 and an in-situ to mean strength ratio of 0.9, which results in a mean in-situ to characteristic strength ratio of 1.08, i.e. $f_{cmi} = 1.08f_c'$. Mean in-situ compressive strength values for concrete grades typically specified in RC walls using the AS 3600 recommendations, the Foster et al. [F1] recommendations and the recommended 1.08 ratio are summarised in Table 4.16.

Figure 4.34: Comparison of mean to characteristic strength ratios for concrete.
Table 4.16: Summary of in-situ compressive strengths for typical concrete grades for RC walls.

<table>
<thead>
<tr>
<th></th>
<th>N32</th>
<th>N40</th>
<th>N50</th>
<th>S65</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS 3600 [X6]</td>
<td>35.4 MPa</td>
<td>43.7 MPa</td>
<td>53.7 MPa</td>
<td>68.2 MPa</td>
</tr>
<tr>
<td>Foster et al. [F1]</td>
<td>34.0 MPa</td>
<td>42.8 MPa</td>
<td>53.2 MPa</td>
<td>70.5 MPa</td>
</tr>
<tr>
<td>Recommended</td>
<td>34.6 MPa</td>
<td>43.2 MPa</td>
<td>54.0 MPa</td>
<td>70.2 MPa</td>
</tr>
</tbody>
</table>

4.5 Experimental Testing of Reinforced Concrete Walls

4.5.1 Review of In-Plane RC Wall Testing in Literature

Over the past 15 years there has been a significant increase in the number of experimental research studies testing the in-plane lateral load capacity of RC walls. A comprehensive literature review was performed to determine the scope of RC wall testing that has been performed locally and abroad. 81 research studies were identified with a total of 506 experimental tests performed on RC walls. These studies were undertaken in various countries and continents including Asia, Australia, Europe, Japan, North America, New Zealand and South America. A summary of all the RC wall tests performed in these studies are presented in Tables 4.17 and 4.18 and show that of the 506 test specimens, 334 were rectangular walls and 172 were non-rectangular walls (e.g. 'T' sections, 'U' sections or rectangular walls with engaged columns at one or both ends of the wall).

The displacement response of RC walls is dependent on many different parameters. Three key parameters have been used to categorise all the test specimens identified in the literature:

1. The shear span ratio (i.e. $M^*/(V^*L_w)$), or alternatively put, the aspect ratio of the equivalent single degree-of-freedom (DOF) system, (i.e. $H_e/L_w$).
2. The type of reinforcement detailing in the wall (i.e. ductile or non-ductile detailing).
3. The cross-sectional shape of the wall (i.e. rectangular or non-rectangular).

For walls with a small aspect ratio (i.e. a squat wall) the displacement of the wall predominately consists of shear deformations. The Eurocode [X10], Canadian [X32] and New Zealand [X8] standards for concrete structures generally recommend shear walls with an aspect ratio less than or equal to 2 are considered squat walls. Walls with an aspect ratio greater than 2 could generally be referred to as slender walls. As the slenderess (i.e. aspect ratio) of the wall increases the displacement response shifts from consisting of predominantly shear deformations to flexural deformations. Paulay and Priestley [P1] propose that the shear deformations in slender cantilever walls with an aspect ratio greater than 4 can be neglected. The RC test specimens in Tables 4.17 and 4.18 have been separated into three different groups of aspect ratios: $H_e/L_w \leq 2$, i.e. shear deformations would dominant the response; $2 < H_e/L_w < 4$, i.e. the response would consist of both shear and flexural deformations; $H_e/L_w \geq 4$, i.e. flexural deformations would dominant the response. The aspect ratio of the wall where the height is taken as the effective height, i.e. the height of the equivalent single DOF system, is equivalent to the shear-span ratio of the wall. The shear-span ratio is equal to the moment at the base of the wall divided by the product of the shear force and wall length, i.e. $M^*/(V^*L_w)$. The shear-span ratio is a more effective way of categorising wall response than the overall height to length ratio (i.e. $H_w/L_w$), as the latter does not consider the type of load distribution that is present on the wall.
Further, the RC test specimens in Tables 4.17 and 4.18 have been separated by the type of reinforcement detailing. Ductile detailing has been broadly defined as walls that have transverse confinement reinforcement in their end regions or boundary elements and non-ductile detailing has been defined as walls that have no close ligatures or boundary elements. Note in this context a boundary element is an engaged column at one or both ends of the wall. An example of ductile and non-ductile detailing can be seen in Figure 3.6. Ductile detailing can dramatically change and increase the displacement response of an RC wall as it provides confinement to the concrete, which increases the ultimate compression strain of the concrete, allowing larger displacements in compress-face controlled walls. It also provides lateral restraint to the tensile reinforcement to prevent local buckling of the vertical reinforcement under reversed cyclic loading.

The cross-sectional shape of a wall greatly affects the behaviour of the wall under lateral load. The response of rectangular walls is generally limited by excessive compressive strains being developed in the extreme compressive fibre of the wall (i.e. compression-face controlled – refer Section 4.3.4.1). The response of isolated box shaped walls, or flanged walls in general, are generally limited by tensile strain limits being developed in the extreme tensile reinforcement of the wall (i.e. tension-face controlled – refer Section 4.3.4.1). The latter is caused by the wall having a very large effective compressive flange, which results in smaller neutral axis depths and as such, the wall can develop very large compressive forces at relatively low compressive strains.

4.5.2 Gaps in In-Plane RC Wall Testing in Literature

It was identified in Chapter 3 that RC walls constructed in Australia typically fall into the category of walls with non-ductile detailing. There has been significantly less research effort directed at testing non-ductile walls, where there have been 143 specimens tested, which is far less than the amount of ductile wall tests, where there have been 363 specimens tested. Similarly, in the case study buildings surveyed only 28% of the walls were rectangular, yet of the 143 non-ductile wall tests identified, 132 consisted of rectangular walls (i.e. 92%).

Further, it was identified in Chapter 3 that the mean range of wall aspect ratios was between 5.0 and 9.8. These values can be adjusted to approximate shear-span ratio values by multiplying them by 0.75, as the effective height of first mode dominant buildings is equal to approximately 75% of the overall height. Resulting in a mean range between 3.75 and 7.35. Chapter 3 also identified that 72% of walls were non-rectangular. This suggests that the majority of RC walls in Australia are non-rectangular walls with shear-span ratios greater than 4. To the author’s best knowledge, there have been no experimental wall tests on non-ductile non-rectangular walls with aspect ratios greater than 4 (refer Table 4.18).

There was one experimental testing program identified that tested non-ductile non-rectangular walls of a moderate aspect ratio [R1]. This test program consisted of two T section non-ductile walls, which are not commonly used wall cross sections in Australia (refer Figure 3.3). These walls were also detailed with one central single layer of horizontal and vertical reinforcement, which is not a common approach for detailing cast in-situ walls in Australia. The most common type of wall cross-section in Australia was observed to be box shaped, bundled box or geometric shaped building cores (refer Figure 3.3), of which no experimental studies were identified for non-ductile construction.
Table 4.17: Number of rectangular RC wall test specimens in literature.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Non-ductile detailing</th>
<th>Ductile detailing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\frac{H_e}{L_w} \leq 2$</td>
<td>$2 &lt; \frac{H_e}{L_w} &lt; 4$</td>
</tr>
<tr>
<td>Cardenas, Russell and Corley [C10]</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>Cardenas and Magura [C11]</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Oesterle, Firorato and Corley [O1]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shiu et al. [S15]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wiradinata and Saatcioglu [W17]</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Elnashai, Pilakoutas and Ambraseys [E4]</td>
<td></td>
<td>9</td>
</tr>
<tr>
<td>Lefas and Kotsovos [L14]</td>
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<td></td>
</tr>
<tr>
<td>Lefas, Kotsovos and Ambraseys [L15]</td>
<td></td>
<td>13</td>
</tr>
<tr>
<td>Yanez, Park and Paulay [Y1]</td>
<td>6</td>
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<tr>
<td>Pilakoutas and Elnashai [P13]</td>
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<td>6</td>
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<tr>
<td>Thomsen and Wallace [T5]</td>
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</tr>
<tr>
<td>Salonikios et al. [S16]</td>
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<td>11</td>
</tr>
<tr>
<td>Tasnimi [T6]</td>
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</tr>
<tr>
<td>Zhang and Wang [Z1]</td>
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<td></td>
</tr>
<tr>
<td>Hidalgo, Ledezma and Jordan [H6]</td>
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</tr>
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<td>Oh, Han and Lee [O2]</td>
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<td>2</td>
</tr>
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<td>Riva, Meda and Giuriani [R4]</td>
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<tr>
<td>Liu [L16]</td>
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</tr>
<tr>
<td>Greifenhagen and Lestuzzi [G9]</td>
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<tr>
<td>Khalil and Ghobarah [K4]</td>
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<td>Kuang and Ho [K5]</td>
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<td>4</td>
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<td>Ireland, Pampanin and Bull [I1]</td>
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<tr>
<td>Lestuzzi and Bachmann [L17]</td>
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<td>Shirai, Matsumori and Kabeyasawa [S17]</td>
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<td>Su and Wong [S18]</td>
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<td>Dazio, Beyer and Bachmann [D3]</td>
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<td></td>
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<tr>
<td>Ghorbanirenani et al. [G10]</td>
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<td></td>
</tr>
<tr>
<td>Orakcal, Massone and Wallace [O3]</td>
<td>14</td>
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<tr>
<td>Zhang, Lu and Wu [Z2]</td>
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<td>2</td>
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### Table 4.17 (cont.): Number of rectangular RC wall test specimens in literature.

<table>
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<tr>
<th>Reference</th>
<th>Non-ductile detailing</th>
<th>Ductile detailing</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>$H_e / L_w \leq 2$</td>
<td>$2 &lt; H_e / L_w &lt; 4$</td>
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<tr>
<td>Huang and Zhao [H7]</td>
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<tr>
<td>Chang, Chang and Lu [C12]</td>
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<tr>
<td>Ghorbanian et al. [G11]</td>
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<td>2</td>
</tr>
<tr>
<td>Lowes et al. [L18]</td>
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<td>4</td>
</tr>
<tr>
<td>Sunely and Kusunoki [S19]</td>
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<tr>
<td>Carrillo and Alcocer [C13]</td>
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<td>Aaleti et al. [A4]</td>
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<td>Smith, Kurama and McGinnis [S20]</td>
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<td>Zhang et al. [Z3]</td>
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<td>Tran and Wallace [T8]</td>
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<td>Albidah [A5] and Altheeb [A6]</td>
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<td>Lu [L8]</td>
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<td>Park et al. [P14]</td>
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<td>Sritharan and Nazari [S21]</td>
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<tr>
<td>Dashti et al. [D1]</td>
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<td>Looi and Su [L19]</td>
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<td>Shegay et al. [S9]</td>
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<td>Puranam and Pujol [P15]</td>
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<tr>
<td>Tran et al. [T9]</td>
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<td>Zhangfeng and Zhengxing [Z4]</td>
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<td>Motter and Wallace [M24]</td>
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<td>Seifi, Henry and Ingham [S22]</td>
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<td>Nakachi [N3]</td>
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<td>Matsui, Saito and Reyna [M25]</td>
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<tr>
<td>Segura and Wallace [S13]</td>
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<tr>
<td><strong>Total number of specimens</strong></td>
<td><strong>96</strong></td>
<td><strong>25</strong></td>
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Table 4.18: Number of non-rectangular RC wall test specimens in literature.

<table>
<thead>
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<th>Reference</th>
<th>Non-ductile detailing</th>
<th>Ductile detailing</th>
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<tbody>
<tr>
<td></td>
<td>$\frac{H_e}{L_w} \leq 2$</td>
<td>$2 &lt; \frac{H_e}{L_w} &lt; 4$</td>
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<td>Barda, Hanson and Corley [B4]</td>
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<tr>
<td>Oesterle et al. [O1]</td>
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<td></td>
</tr>
<tr>
<td>Maier and Thulimann [M26]</td>
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<td></td>
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<tr>
<td>Thomsen and Wallace [T5]</td>
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<td>Gupta and Rangan [G13]</td>
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<td>Kabeyasawa and Hiraishi [K6]</td>
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<td>Mo and Kuo [M27]</td>
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<td>Hines, Seible and Priestley [H9]</td>
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<td>Oh et al. [O2]</td>
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<td>Palermo and Vecchio [P16]</td>
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<tr>
<td>Dabbagh [D4]</td>
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<td>Ile and Reynouard [I2]</td>
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<td>Beyer, Dazio and Priestley [B5]</td>
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<td>Farvashany, Foster and Rangan [F2]</td>
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<td>Brueggen [B6]</td>
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<td>Karamlou and Kabir [K7]</td>
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<td>Burgueno, Liu and Hines [B7]</td>
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<td>Taleb et al. [T7]</td>
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<td>Park et al. [P14]</td>
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<td>Li, Gian and Wu [L22]</td>
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<tr>
<td>Rosso et al. [R1]</td>
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<tr>
<td>Lim, Kang and Hong [L23]</td>
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<td>Maeda et al. [M28]</td>
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<td>Correal et al. [C14]</td>
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<td>Segura and Wallace [S13]</td>
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</tr>
<tr>
<td><strong>Total number of specimens</strong></td>
<td><strong>9</strong></td>
<td><strong>2</strong></td>
</tr>
</tbody>
</table>
4.6 Conclusions

There is little research or guidance available in literature for assessing the displacement behaviour of limited ductile RC walls. The majority of guidance provided in literature is for ductile RC walls, which are generally detailed to have well confined end regions and no lap splices of the vertical reinforcement in the plastic hinge region. Meaning that the guidance that is available, is likely not relevant to the majority of RC wall buildings constructed or currently being designed and constructed in Australia.

Failure mechanisms of RC walls include flexure, shear, out-of-plane buckling and local bar buckling of the vertical reinforcement. Out-of-plane buckling and local bar buckling result after significant inelastic tensile strains are developed in the vertical reinforcement in the previous reversed cyclic load cycle, resulting in large residual crack widths across the end region of the wall. On load reversal the wall undergoes a bifurcation effect where it can either ‘over-come’ these inelastic tensile strains in compression, therefore closing the residual crack widths and allowing the end region of the wall to regain its initial axial stiffness and behave as expected; or the wall cannot close these residual cracks and the reduced axial stiffness in the end region of the wall allows an out-of-plane buckling or local bar buckling failure mechanism to develop.

Flexure failure of the wall is either compression-face or tension-face controlled. Compression-face controlled failure is when the lateral displacement of the wall is limited by the maximum concrete compressive strain being reached in the extreme concrete compressive fibre, whereas tension-face controlled failure is when it is limited by the maximum reinforcement tensile strain being reached in the extreme tensile reinforcing bar. Limited ductile rectangular walls are usually compression-face controlled, whereas walls with large compression flanges (e.g. building cores) are usually tension-face controlled since the large compression flange allows large compression forces to be developed at relatively low compressive strain levels, which results in smaller neutral axis depths and larger tensile strains in the reinforcement.

A preliminary parametric study into rectangular limited ductile RC walls was performed to see how various parameters (e.g. wall length, concrete grade, vertical reinforcement ratio, axial load ratio) affect the displacement behaviour and inelastic response. The axial load ratio was shown to be the most dominant parameter that affects the force-displacement behaviour.

Mean material strengths for standard grades of reinforcement and concrete in Australia that can be utilised for performing non-linear analyses of RC elements or structures have been recommended. The recommended reinforcement properties were made using a large database of tensile test data that included 285 rebar tensile tests performed at Swinburne University of Technology and another 6,257 rebar tensile tests performed by an independent material testing laboratory, whom are contracted by industry suppliers to assess code compliance of their reinforcing bars. The database included test data from a 7-year period from 2011 to 2017.

A significant amount of experimental testing programs into the lateral displacement behaviour of RC walls have been performed in the last 15 years, particularly following the 2010 Chile and 2011 Christchurch earthquakes, where many examples of poor performances in RC walls were observed. Fortunately, very rarely did these instances result in complete structural collapse of the building. The majority of these testing programs are unfortunately focused at the lateral performance of ductile RC walls and hence not directly relevant to Australian construction practices. This represents a large gap in literature for the experimental testing of limited ductile RC walls and components.
Chapter 5

Experimental Testing of RC Wall Boundary Elements

This chapter presents an overview, results and discussions on the first program of large-scale experimental testing performed on RC walls. The first program of experimental work consisted of seventeen boundary element test specimens. The boundary element test specimens represented the end region of an RC wall and were used to experimentally study different failure mechanisms of RC walls and how various detailing techniques affected the behaviour. The failure mechanisms of interest were global out-of-plane buckling and local bar buckling of the vertical reinforcement. The specimens included both high and low slenderness ratios (i.e. $H_w/t_w$) and both cast in-situ and precast wall detailing. The test specimens were designed and detailed to best match standard industry practices (as documented in Chapter 3). The testing was performed using various test machines in the Smart Structures Laboratory at Swinburne University of Technology. This chapter also presents proposed strain limits for limited ductile walls, which were determined with respect to recent best-practice advice for RC construction in regions of higher seismicity and the results of the boundary element test specimen results. Finally, the chapter is concluded with a tension stiffening model for limited ductile RC elements that was developed using the boundary element test specimens documented at the beginning of the chapter.

5.1 Experimental Testing Plan

This experimental testing program consisted of seventeen boundary element test specimens. The specimens represented the end region of RC wall or building core, which under cyclic lateral loads, is essentially subjected to cyclic tension-compression axial load, as illustrated in Figure 5.1. This has become a popular form of experimental testing in recent years (e.g. [H10, P17, R2, S23, T12, W18]) and is commonly referred to as ‘prism testing’. This form of testing allows for the performance of various RC wall end region detailing techniques to be experimentally assessed without having to perform costly full-scale RC wall tests, such as those documented in Chapter 6.
Figure 5.1: Configurations and failure mechanisms of boundary element test specimens.
The experimental testing program consisted of seventeen prism specimens. The specimens in this experimental program differed from other experimental studies in literature, which typically consisted of well detailed ductile prisms. The prism specimens in this experimental program were designed to represent the end regions of limited ductile walls.

The first six specimens (i.e. P01 to P06) were ‘type 1’ specimens, which were higher slenderness ratio (i.e. $H_w/t_w$) boundary elements. The other eleven specimens (i.e. P07 to P17) were ‘type 2’ specimens, which were lower slenderness ratio boundary elements. The failure mechanisms of interest for the type 1 and type 2 specimens was primarily global out-of-plane buckling and local bar buckling respectively (refer Figure 5.1). Different reinforcement ratios and configurations were considered for both types of specimens. The type 1 specimens had vertical reinforcement ratios varying from 0.6% to 2.1% and similarly, the type 2 specimens had vertical reinforcement ratios varying from 0.4% to 2.1%. Both types had specimens with reinforcement ‘each face’ (i.e. vertical and horizontal bars on each face of the specimen) and ‘central’ (i.e. one layer of vertical and horizontal bars located centrally in the specimen). The prism specimen test matrix is shown in Table 5.1 and construction drawings for each test specimen are provided in Appendix A.

The details of the type 1 and type 2 specimens are presented in the subsequent two sub-sections respectively. The type 2 specimens were tested in two different testing programs (i.e. round one and round two). The type 2 round one specimens were constructed without any splicing of the vertical reinforcement, similar to the type 1 specimens. Whereas, the type 2 round two specimens were constructed with either lap splices or precast grout tube connections at the base of the prism.

**Table 5.1: Prism specimen test matrix.**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$H_w/t_w$</th>
<th>Reinf. layout</th>
<th>Vertical reinf.</th>
<th>Reinf. ratio</th>
<th>Splice detail</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type 1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P01</td>
<td>15.4</td>
<td>each face</td>
<td>6-N12</td>
<td>0.012</td>
<td>none</td>
</tr>
<tr>
<td>P02</td>
<td>15.4</td>
<td>each face</td>
<td>6-N12</td>
<td>0.012</td>
<td>none</td>
</tr>
<tr>
<td>P03</td>
<td>15.4</td>
<td>each face</td>
<td>6-N16</td>
<td>0.021</td>
<td>none</td>
</tr>
<tr>
<td>P04</td>
<td>15.4</td>
<td>each face</td>
<td>6-N16</td>
<td>0.021</td>
<td>none</td>
</tr>
<tr>
<td>P05</td>
<td>15.4</td>
<td>central</td>
<td>3-N12</td>
<td>0.006</td>
<td>none</td>
</tr>
<tr>
<td>P06</td>
<td>15.4</td>
<td>central</td>
<td>3-N12</td>
<td>0.006</td>
<td>none</td>
</tr>
<tr>
<td><strong>Type 2</strong> (round 1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P07</td>
<td>6.2</td>
<td>each face</td>
<td>6-N10</td>
<td>0.008</td>
<td>none</td>
</tr>
<tr>
<td>P08</td>
<td>6.2</td>
<td>each face</td>
<td>6-N16</td>
<td>0.021</td>
<td>none</td>
</tr>
<tr>
<td>P09</td>
<td>6.2</td>
<td>central</td>
<td>3-N10</td>
<td>0.004</td>
<td>none</td>
</tr>
<tr>
<td>P10</td>
<td>6.2</td>
<td>central</td>
<td>3-N16</td>
<td>0.010</td>
<td>none</td>
</tr>
<tr>
<td><strong>Type 2b</strong> (round 2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P11</td>
<td>2.7</td>
<td>each face</td>
<td>6-N10</td>
<td>0.007</td>
<td>lap splice</td>
</tr>
<tr>
<td>P12</td>
<td>2.7</td>
<td>each face</td>
<td>6-N10</td>
<td>0.010</td>
<td>lap splice</td>
</tr>
<tr>
<td>P13</td>
<td>2.8</td>
<td>each face</td>
<td>4-N10</td>
<td>0.005</td>
<td>lap splice</td>
</tr>
<tr>
<td>P14</td>
<td>2.8</td>
<td>each face</td>
<td>6-N16</td>
<td>0.018</td>
<td>lap splice</td>
</tr>
<tr>
<td>P15</td>
<td>2.8</td>
<td>each face</td>
<td>8-N12</td>
<td>0.013</td>
<td>grout tube</td>
</tr>
<tr>
<td>P16</td>
<td>2.8</td>
<td>each face</td>
<td>6-N12</td>
<td>0.010</td>
<td>grout tube</td>
</tr>
<tr>
<td>P17</td>
<td>2.8</td>
<td>each face</td>
<td>6-L11.9</td>
<td>0.010</td>
<td>grout tube</td>
</tr>
</tbody>
</table>
5.1.1 Type 1 Specimens – P01 to P06

All six type 1 specimens were 2000 mm tall, 450 mm long and 130 mm thick. This meant the slenderness ratio for the specimens was 15.4, which was close to the upper bound mean value identified in the reconnaissance survey performed in Chapter 3 (refer Table 3.1). The first two specimens, i.e. P01 and P02, had a ‘moderate’ vertical reinforcement content of 1.2%, which consisted of 6-N12 vertical bars with 3 bars on each face of the wall. The second two specimens, i.e. P03 and P04, had a ‘heavy’ vertical reinforcement content of 2.1%, which consisted of 6-N16 vertical bars with 3 bars on each face of the wall. The last two specimens, i.e. P05 and P06, had a ‘light’ vertical reinforcement content of 0.6%, which consisted of 3-N12 vertical bars, however these three bars were placed centrally in the wall, unlike P01 to P04. Bar sizes N12 and N16 denote 12 mm and 16 mm nominal diameter grade D500N reinforcing bars to AS/NZS 4671 [X29] respectively. Grade D500N denotes deformed ‘normal ductility’ reinforcement with a characteristic yield stress of 500 MPa. The properties of each test specimen are summarised in Table 5.2 and illustrated in Figure 5.2.

The specimens represented a 450 mm long segment of a longer section of wall. Because the specimens were not long enough to ensure the horizontal reinforcement would be sufficiently developed, the horizontal reinforcement was substituted for all-thread (i.e. threaded rod) with a nut at each end, as shown in Figure 5.2. The nut at each end ensures the all-thread is full developed, hence providing the equivalent amount of unidirectional confinement that would be present in an equivalent longer section of wall. Test specimens P01 to P04 were detailed using two grids of vertical and horizontal bars. Whereas test specimens P05 and P06 were detailed using one central grid of vertical and horizontal bars. The horizontal bars for all six specimens consisted of M8 all-threads at 200 centres, resulting in horizontal reinforcement ratios of 0.28% for P01 to P04 and 0.14% for P05 and P06.

Table 5.2: Test specimen properties (boundary element specimens P01 to P06).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Thickness (mm)</th>
<th>Length (mm)</th>
<th>Height (mm)</th>
<th>( \frac{H_w}{t_w} )</th>
<th>Vertical reinf.</th>
<th>Reinf. ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>P01</td>
<td>130</td>
<td>450</td>
<td>2000</td>
<td>15.4</td>
<td>6-N12</td>
<td>0.012</td>
</tr>
<tr>
<td>P02</td>
<td>130</td>
<td>450</td>
<td>2000</td>
<td>15.4</td>
<td>6-N12</td>
<td>0.012</td>
</tr>
<tr>
<td>P03</td>
<td>130</td>
<td>450</td>
<td>2000</td>
<td>15.4</td>
<td>6-N16</td>
<td>0.021</td>
</tr>
<tr>
<td>P04</td>
<td>130</td>
<td>450</td>
<td>2000</td>
<td>15.4</td>
<td>6-N16</td>
<td>0.021</td>
</tr>
<tr>
<td>P05</td>
<td>130</td>
<td>450</td>
<td>2000</td>
<td>15.4</td>
<td>3-N12</td>
<td>0.006</td>
</tr>
<tr>
<td>P06</td>
<td>130</td>
<td>450</td>
<td>2000</td>
<td>15.4</td>
<td>3-N12</td>
<td>0.006</td>
</tr>
</tbody>
</table>

The specimens were constructed using standard N40 grade concrete with a minimum characteristic 28-day compressive cylinder strength of 40 MPa and a maximum aggregate size of 14 mm. The compressive strength of the concrete was determined on test day using 100 mm diameter and 200 mm long cylinder tests. The tensile strength was indirectly determined from the test specimen itself by dividing the force observed when cracking first occurred by the net area of the concrete, i.e. the gross area of concrete minus the area of the vertical bars. This approach for calculating the tensile strength does not necessarily yield the ‘true’ material strength as it can be affected by shrinkage or the test setup of the specimen. It should more correctly be referred to as the ‘pseudo tensile strength’. The results are summarised in Table 5.3.
Figure 5.2: Test specimens P01 to P06 details.

Table 5.3: Test specimen concrete strength (boundary element specimens P01 to P06).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specified strength</th>
<th>Actual strength</th>
<th>Pseudo tensile strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>P01</td>
<td>40 MPa</td>
<td>44.7 MPa</td>
<td>2.4 MPa</td>
</tr>
<tr>
<td>P02</td>
<td>40 MPa</td>
<td>44.7 MPa</td>
<td>2.4 MPa</td>
</tr>
<tr>
<td>P03</td>
<td>40 MPa</td>
<td>53.5 MPa</td>
<td>2.7 MPa</td>
</tr>
<tr>
<td>P04</td>
<td>40 MPa</td>
<td>56.2 MPa</td>
<td>NA</td>
</tr>
<tr>
<td>P05</td>
<td>40 MPa</td>
<td>53.5 MPa</td>
<td>NA</td>
</tr>
<tr>
<td>P06</td>
<td>40 MPa</td>
<td>53.5 MPa</td>
<td>NA</td>
</tr>
</tbody>
</table>

Test specimens P01 and P02 were poured and tested on the same day and as such have the same concrete properties. Similarity, P05 and P06 were also poured and tested on the same day as each other and have the same concrete properties. Test specimens P03 and P04 were also poured on the same day, however they were tested on different days, meaning the concrete properties were different. Interestingly, P04 was tested approximately 600 days after P03, however there was only a marginal compressive strength gain of 5%. The tensile strength of P04 was not calculated.
because initial cracking of the specimen occurred when the specimen was being setup in the test machine due to a malfunction in the machine that resulted in a small tension load being applied to the specimen. This occurred while the data acquisition was not being used. The tension load applied did not cause any yielding of the vertical reinforcement. As such it was determined that the malfunction did not affect the subsequent behaviour of P04 during the test. Further, there was a system error during the beginning of the test for specimen S05 (which is discussed further in Section 5.3.1.2) that meant the pseudo tensile strength for P05 and P06 could not be determined.

Reinforcement material samples for test specimens P01 to P06 were not taken. The yield stress (i.e. $f_{sy}$) of the bars was indirectly calculated from the force-displacement response of the respective test specimen (where possible). The author notes that these will not be the ‘true’ yield strength values of the vertical reinforcement, however it does provide a general estimate of what the average strength of the vertical reinforcement in the specimen actually was. The ultimate stress (i.e. $f_{su}$) and ultimate strain (i.e. $\varepsilon_{su}$) is unknown. Each pair of specimens (i.e. P01/P02, P03/P04 and P05/P06) were constructed using the same batch of reinforcement. The pseudo reinforcement properties are summarised in Table 5.4.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement</th>
<th>$f_{sy}$*</th>
<th>$f_{su}$</th>
<th>$f_{sy}/f_{su}$</th>
<th>$\varepsilon_{su}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P01</td>
<td>N12 vertical bars</td>
<td>590 MPa</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>P02</td>
<td>N12 vertical bars</td>
<td>590 MPa</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>P03</td>
<td>N16 vertical bars</td>
<td>612 MPa</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>P04</td>
<td>N16 vertical bars</td>
<td>612 MPa</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>P05</td>
<td>N12 vertical bars</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>P06</td>
<td>N12 vertical bars</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

* These material properties are pseudo strengths due to the way they were calculated. Refer to the in-text discussion for further details.

5.1.2 Type 2 Specimens – P07 to P17

The experimental work for the type 2 test specimens were conducted in two different rounds of testing. The first round of testing consisted of test specimens P07 to P10. These specimens, along with the type 1 specimens (i.e. P01 to P06), were manufactured by a local precast company in Melbourne. The second round of testing consisted of test specimens P11 to P17, which were manufactured by the author. The first and second round type 2 test specimens are presented in the following two sub-sections, i.e. Sections 5.1.2.1 and 5.1.2.2 respectively.

5.1.2.1 Type 2 Specimens – Round One Testing – P07 to P10

The first round of type 2 specimens (i.e. P07 to P10) were essentially the same as the type 1 test specimens (i.e. P01 to P06), except they were 800 mm tall, which the slenderness ratio was 6.2 or 2.5 times smaller than the type 1 specimens. The specimens were constructed using similar vertical reinforcement ratios to the type 1 specimens and also consisted of both specimens with single central layers of vertical and horizontal reinforcement and specimens with grids vertical and horizontal on each face of the wall. The specimens used the same configuration of horizontal reinforcement as the type 1 specimens (i.e. M8 all-threads with nuts at each end). The properties of each test specimen are summarised in Table 5.5 and illustrated in Figure 5.3.
Table 5.5: Test specimen properties (boundary element specimens P07 to P10).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Thickness (mm)</th>
<th>Length (mm)</th>
<th>Height (mm)</th>
<th>( H_w )</th>
<th>Vertical reinf.</th>
<th>Reinf. ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>P07</td>
<td>130</td>
<td>450</td>
<td>800</td>
<td>6.2</td>
<td>6-N10</td>
<td>0.008</td>
</tr>
<tr>
<td>P08</td>
<td>130</td>
<td>450</td>
<td>800</td>
<td>6.2</td>
<td>6-N16</td>
<td>0.021</td>
</tr>
<tr>
<td>P09</td>
<td>130</td>
<td>450</td>
<td>800</td>
<td>6.2</td>
<td>3-N10</td>
<td>0.004</td>
</tr>
<tr>
<td>P10</td>
<td>130</td>
<td>450</td>
<td>800</td>
<td>6.2</td>
<td>3-N16</td>
<td>0.010</td>
</tr>
</tbody>
</table>

Figure 5.3: Test specimens P07 to P10 details.

The four specimens had a crack propagator at mid-height, as shown in Figure 5.3. Strain gauges were located at the mid-height of the wall and the crack propagator was used to ensure a crack occurred at this location where the gauges were located.

The specimens were constructed using the same concrete mix as the type 1 specimens (i.e. P01 to P06), which was a N40 grade concrete and is discussed above in Section 5.1.1. The compressive strength of the concrete was determined on test day using 100 mm diameter and 200 mm long cylinder tests. The tensile strengths were indirectly determined from the test specimens using the same procedure as the type 1 specimens. This approach results in a ‘pseudo tensile strength’ of the concrete, rather than its ‘true’ tensile strength (as discussed in Section 5.1.1). The results are summarised in Table 5.6.
Table 5.6: Test specimen concrete strength (boundary element specimens P07 to P10).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specified strength</th>
<th>Actual strength</th>
<th>Pseudo tensile strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>P07</td>
<td>40 MPa</td>
<td>45.4 MPa</td>
<td>2.9 MPa</td>
</tr>
<tr>
<td>P08</td>
<td>40 MPa</td>
<td>45.4 MPa</td>
<td>2.9 MPa</td>
</tr>
<tr>
<td>P09</td>
<td>40 MPa</td>
<td>43.2 MPa</td>
<td>3.1 MPa</td>
</tr>
<tr>
<td>P10</td>
<td>40 MPa</td>
<td>43.2 MPa</td>
<td>2.6 MPa</td>
</tr>
</tbody>
</table>

Reinforcement material samples for test specimens P07 to P10 were not taken. Similar to the type 1 tests specimens, the reinforcement material properties were indirectly calculated using the force-displacement response of the respective test specimen and the strain gauges (as discussed in Section 5.2.1) that were attached to the vertical bars. The N10 bars in specimen P07 and P10 and the N16 bars in specimen P08 and P10 were sourced from the same respective batches of reinforcement. The specimens were constructed using grade D500N bars in accordance with AS/NZS 4671. The pseudo reinforcement properties are summarised in Table 5.7.

Table 5.7: Test specimen reinforcement properties (boundary element specimens P07 to P10).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement</th>
<th>( f_{sy} )*</th>
<th>( f_{su} )*</th>
<th>( f_{sy}/f_{su} )</th>
<th>( \varepsilon_{su} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>P07</td>
<td>N10 vertical bars</td>
<td>593</td>
<td>709</td>
<td>1.20</td>
<td>NA</td>
</tr>
<tr>
<td>P08</td>
<td>N16 vertical bars</td>
<td>600</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>P09</td>
<td>N10 vertical bars</td>
<td>593</td>
<td>709</td>
<td>1.20</td>
<td>NA</td>
</tr>
<tr>
<td>P10</td>
<td>N16 vertical bars</td>
<td>600</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

* These material properties are pseudo strengths due to the way they were calculated. Refer to the in-text discussion for further details.

5.1.2.2 Type 2 Specimens – Round Two Testing – P11 to P17

The second round of type 2 specimens (i.e. P11 to P17) were somewhat different from the first round of specimens (i.e. P07 to P10). The specimens were still 450 mm long, however the thickness was increased to 150 mm and the height was either 800 and 850 mm, depending on the specimen. The first four specimens (i.e. P11 to P14) were cast-in situ style specimens (similar to P01 to P10), while the last three specimens (i.e. P15 to P17) were precast style specimens. The cast in-situ style specimens had a lap splice at the base of the wall whereas the precast style specimens had a grout tube connection, as shown in Figure 5.4. The properties of each test specimen are summarised in Table 5.8.

The lap splice length for the cast in-situ style specimens was calculated in accordance with AS 3600 [X6], which is a function of: the yield stress of the bar; the characteristic compressive strength of the concrete; the bar diameter; and either the clear cover or the vertical bar spacing. The lap splice lengths for the different reinforcement layouts and bar sizes in specimens P11 to P14 are shown in Figure 5.4. However, for the specimens with a grout tube connection, AS 3600 does not offer a ‘codified’ procedure for calculating the required dowel embedment length. One method, which many designers in industry adopt, for calculating the required dowel embedment length is to take the greater of the length required to bond the dowel bar to the cementitious grout in the grout tube and the length required to bond the corrugated grout tube to the surrounding concrete (assuming the dowel bar has a larger diameter than the main vertical bars of the wall).
The length required to bond the dowel bar to the cementitious grout in the grout tube is calculated using the development length equations in AS 3600, however because the dowel bar is within the grout tube, it can be argued that the grout tube itself provides transverse confinement pressure, allowing the refined development length procedure to be adopted. This allows a shorter lap length than would otherwise be required to be adopted. Similarly, the length required to bond the corrugated grout tube to the sounding concrete is calculated using the development length equations in AS 3600 (i.e. Equation 5.1), however the bar diameter (i.e. $d_b$) is substituted for the grout tube diameter (i.e. $d_g$) and the yield stress (i.e. $f_{sy}$) is substituted for a value equal to the yield stress of the dowel bar multiplied by the ratio of dowel bar diameter squared divided by the grout tube diameter squared, i.e. $f_{sy} \times \left(\frac{d_b^2}{d_g^2}\right)$. The greater of these two lengths is then multiplied by 1.25 and taken as the dowel embedment. The 1.25 factor is specified for tensile lap lengths in AS 3600.

$$L_{sy.tb} = \frac{0.5k_1k_2f_{sy}d_b}{k_2\sqrt{f'_c}} \geq 29k_1d_b \quad \ldots 5.1$$

Where:

- $k_1, k_2, k_3 = \text{factors that account for the reinforcing bar size and configuration adopted}$
- $d_b = \text{bar diameter}$
- $f_{sy} = \text{yield stress of bar}$
- $f'_c = \text{concrete characteristic compressive strength}$

It is questionable whether this approach, which is commonly used by designer engineers in both Australia and New Zealand, is actually 'code compliant' to either the Australian or New Zealand concrete standards. There should likely be some allowance given to the fact that these connections are actually non-contact splices and, in some cases, it might be more appropriate to check the load path of these connections using strut and tie methods.

The dowel embedment length for all three precast style specimens was taken to be 500 mm. The first precast style specimen (i.e. P15) was designed so the dowel connection was the ‘weak link’. This specimen had 3-N16 dowel bars and the dowel bar embedment length of 500 mm was calculated with respect to this specimen. The second and third precast style specimens (i.e. P16 and P17 respectively) were designed so the vertical bars in the section of wall above the dowel connection were the ‘weak link’. For these specimens, the same dowel bar embedment length as P15 was used, however 4-N20 dowel bars were adopted.

The specimens were constructed using the same concrete mix as the type 1 specimens (i.e. P01 to P06), which was a N40 grade concrete and is discussed above in Section 5.1.1. The compressive strength of the concrete was determined on test day using 100 mm diameter and 200 mm long cylinder tests. The tensile strengths were indirectly determined from the test specimens using the same procedure as the type 1 specimens. This approach results in a ‘pseudo tensile strength’ of the concrete, rather than its ‘true’ tensile strength (as discussed in Section 5.1.1). The results are summarised in Table 5.9.
Figure 5.4: Test specimens P11 to P17 details.
Table 5.8: Test specimen properties (boundary element specimens P11 to P17).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Thickness (mm)</th>
<th>Length (mm)</th>
<th>Height (mm)</th>
<th>$\frac{H_w}{t_w}$</th>
<th>Vertical reinf.</th>
<th>Reinf. ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>P11</td>
<td>150</td>
<td>450</td>
<td>800</td>
<td>2.7*</td>
<td>6-N10</td>
<td>0.007</td>
</tr>
<tr>
<td>P12</td>
<td>150</td>
<td>450</td>
<td>800</td>
<td>2.7*</td>
<td>6-N12</td>
<td>0.010</td>
</tr>
<tr>
<td>P13</td>
<td>150</td>
<td>450</td>
<td>850</td>
<td>2.8*</td>
<td>4-N10</td>
<td>0.005</td>
</tr>
<tr>
<td>P14</td>
<td>150</td>
<td>450</td>
<td>850</td>
<td>2.8*</td>
<td>6-N16</td>
<td>0.018</td>
</tr>
<tr>
<td>P15</td>
<td>150</td>
<td>450</td>
<td>850</td>
<td>2.8*</td>
<td>8-N12</td>
<td>0.013</td>
</tr>
<tr>
<td>P16</td>
<td>150</td>
<td>450</td>
<td>850</td>
<td>2.8*</td>
<td>6-N12</td>
<td>0.010</td>
</tr>
<tr>
<td>P17</td>
<td>150</td>
<td>450</td>
<td>850</td>
<td>2.8*</td>
<td>6-L11.9</td>
<td>0.010</td>
</tr>
</tbody>
</table>

* Test specimens P11 to P17 were tested with a mid-height out-of-plane roller support, as discussed later in Section 5.2.2, which meant the slenderness ratio of the wall was equal to $0.5 \times \left(\frac{H_w}{t_w}\right)$.

Unlike the first ten specimens (i.e. P01 to P10), which were manufactured by a local precaster in Melbourne, specimens P11 to P17 were manufactured by the author at Swinburne University of Technology. The specimens were poured on two different days. Specimens P12, P14, P15 and P16 were poured on the first day and specimens P11, P13 and P17 were poured on the second day. The concrete was ordered from the same local concrete supplier which the precaster used, however because a maximum of only four specimens were being poured on a single day, a ‘mini mix’ was ordered due to the relatively small volume of concrete required. The mini mix ordered had approximately 1.5 cubic metres of concrete, which is significantly less than a standard concrete truck which usually has 8 to 9 cubic meters of concrete. Due to the relatively small size of the concrete mix, small variations in the amount of cement or water added at the batch plant can significant effect the final 28-day compressive strength of the concrete. This can be observed in Table 5.9, where the specimens in the first pour were significantly weaker than the 28-day target strength of 40 MPa.

Table 5.9: Test specimen concrete strength (boundary element specimens P11 to P17).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specified strength</th>
<th>Actual strength</th>
<th>Pseudo tensile strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>P11</td>
<td>40 MPa</td>
<td>45.3 MPa</td>
<td>3.0 MPa</td>
</tr>
<tr>
<td>P12</td>
<td>40 MPa</td>
<td>33.0 MPa</td>
<td>2.1 MPa</td>
</tr>
<tr>
<td>P13</td>
<td>40 MPa</td>
<td>41.1 MPa</td>
<td>1.7 MPa</td>
</tr>
<tr>
<td>P14</td>
<td>40 MPa</td>
<td>33.6 MPa</td>
<td>1.0 MPa</td>
</tr>
<tr>
<td>P15</td>
<td>40 MPa</td>
<td>33.5 MPa</td>
<td>2.4 MPa</td>
</tr>
<tr>
<td>P16</td>
<td>40 MPa</td>
<td>33.6 MPa</td>
<td>1.7 MPa</td>
</tr>
<tr>
<td>P17</td>
<td>40 MPa</td>
<td>41.1 MPa</td>
<td>2.3 MPa</td>
</tr>
</tbody>
</table>

Unlike the previous ten test specimens (i.e. P01 to P10), test specimens P11 to P17 were constructed using N10 closed ligatures at 200 mm centres (as shown in Figure 5.4), as opposed to the M8 all-threads with nuts at each end.
Reinforcement material samples were taken for all the bars in test specimens P11 to P17. The reinforcement material properties were calculated from tensile test of the material samples and are summarised in Table 5.10. The specimens were constructed using grade D500N bars in accordance with AS/NZS 4671, except test specimen P17, which was constructed using grade D500L vertical bars. Grade D500L denotes deformed 'low ductility' reinforcement with a characteristic yield stress of 500 MPa. Stress-strain curves for the reinforcement samples taken are presented in Appendix B.

Table 5.10: Test specimen reinforcement properties (boundary element specimens P11 to P17).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement</th>
<th>$f_{sy}$ (MPa)</th>
<th>$f_{su}$ (MPa)</th>
<th>$f_{sy}/f_{su}$</th>
<th>$\varepsilon_{su}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P11</td>
<td>N10 vertical bars</td>
<td>558</td>
<td>690</td>
<td>1.24</td>
<td>8.5%</td>
</tr>
<tr>
<td>P12</td>
<td>N12 vertical bars</td>
<td>581</td>
<td>656</td>
<td>1.13</td>
<td>7.5%</td>
</tr>
<tr>
<td>P13</td>
<td>N10 vertical bars</td>
<td>561</td>
<td>693</td>
<td>1.24</td>
<td>8.2%</td>
</tr>
<tr>
<td>P14</td>
<td>N16 vertical bars</td>
<td>606</td>
<td>688</td>
<td>1.14</td>
<td>9.0%</td>
</tr>
<tr>
<td>P15</td>
<td>N12 vertical bars</td>
<td>569</td>
<td>648</td>
<td>1.14</td>
<td>8.2%</td>
</tr>
<tr>
<td>P15</td>
<td>N16 dowel bars</td>
<td>607</td>
<td>694</td>
<td>1.14</td>
<td>8.4%</td>
</tr>
<tr>
<td>P16</td>
<td>N12 vertical bars</td>
<td>572</td>
<td>651</td>
<td>1.14</td>
<td>7.7%</td>
</tr>
<tr>
<td>P16</td>
<td>N20 dowel bars</td>
<td>550</td>
<td>643</td>
<td>1.17</td>
<td>10.8%</td>
</tr>
<tr>
<td>P17</td>
<td>L11.9 vertical bars</td>
<td>577</td>
<td>626</td>
<td>1.09</td>
<td>3.2%</td>
</tr>
<tr>
<td>P17</td>
<td>N20 dowel bars</td>
<td>550</td>
<td>644</td>
<td>1.17</td>
<td>10.8%</td>
</tr>
</tbody>
</table>

5.2 Instrumentation, Test Setup and Loading Protocol

5.2.1 Instrumentation

A combination of physical instrumentation attached to the tests specimens and a contactless photogrammetry system was used to monitor and measure the behaviour and response of the test specimens. The physical instrumentation varied per test series, but generally consisted of: string potentiometers, which were used to measure the overall vertical displacement and out-of-plane movement; linear variable displacement transducers (LVDTs), which were also used to measure the overall vertical displacement and out-of-plane displacements; and linear potentiometers, which were used to measure crack widths.

The photogrammetry system used was the V-STARS N series by Geodetic Systems. This system is a turnkey single camera photogrammetry system, which can be used to make discrete measurements of the test specimen while the testing procedure is paused. The photogrammetry system is further discussed in Section 6.2. The location of photogrammetry targets, where the x-y-z movement was calculated and recorded at discrete moments throughout the test, for test specimens P07 to P17, is shown in Figure 5.5.

Post yield strain gauges were also attached to the vertical reinforcement on test specimens P07 to P17. The gauges were located at mid-height for test specimens P07 to P10, i.e. at the crack propagator. For test specimens P11 to P17 they were located at the top and bottom of the lap splice and grout tube connection. The gauges on test specimens P11 to P17 did not produce usable results, typically because the horizontal cracks did not align with where they were located.
5.2.2 Test Setup

The specimens were tested in the Smart Structures Laboratory (SSL) at Swinburne University of Technology. All seventeen test specimens were tested in the MTS 1MN uniaxial test machine, except test specimens P01 and P05, which were tested in the Instron 5MN uniaxial test machine. The MTS 1MN uniaxial test machine is an off-the-shelf testing machine that can apply uniaxial loading with a force capacity of ±1,000 kN and has a stroke length of ±500 mm. Similarly, the Instron 5MN uniaxial test machine is an off-the-shelf testing machine that can apply uniaxial loading with a force capacity of ±5,000 kN and has a stroke length of ±500 mm. The test specimen setup in the respective test machine is presented in Figure 5.6.

Figure 5.5: Test specimens P07 to P17 photogrammetry targets (approximate locations).
Figure 5.6: Boundary element test setup in the 5 MN Instron and 1 MN MTS test machines.
The second round of type 2 test specimens (i.e. P11 to P17) were tested with an out-of-plane roller support at mid-height, as shown in Figure 5.6(d). This was used to prevent any possible out-of-plane buckling failure mechanisms developing, as this was the failure mechanism observed in test specimen P08, which was tested in the first round of type 2 test specimens. The primary failure mechanism of interest for this series of test specimens was local bar buckling of the vertical reinforcement, so the test setup was modified to include this out-of-plane roller support.

### 5.2.3 Loading Protocol

Two different primary types of loading protocols were adopted for testing the boundary element test specimens. The first type of loading protocol was a quasi-static monotonically increasing axial compression load, which was performed in the 5 MN Instron test machine, refer Figure 5.6(b). The second type of loading protocol was cyclic axial tension-compression loading, which was performed in the 1 MN MTS test machine, refer Figures 5.6(a), 5.6(c) and 5.6(d). Further, the cyclic axial tension-compression loading was applied as a quasi-static loading protocol for all the specimens, except test specimen P04, where it was applied as a quasi-dynamic loading protocol. The latter was used to assess whether loading strain rate affects the out-of-plane buckling behaviour of RC walls. The loading protocols used for the seventeen boundary element test specimens is summarised in Table 5.11.

**Table 5.11:** Loading protocol summary – test specimens P01 to P17.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Loading protocol</th>
</tr>
</thead>
<tbody>
<tr>
<td>P01</td>
<td>Quasi-static monotonic axial compression</td>
</tr>
<tr>
<td>P02</td>
<td>Quasi-static cyclic axial tension-compression</td>
</tr>
<tr>
<td>P03</td>
<td>Quasi-static cyclic axial tension-compression</td>
</tr>
<tr>
<td>P04</td>
<td>Quasi-dynamic cyclic axial tension-compression</td>
</tr>
<tr>
<td>P05</td>
<td>Quasi-static monotonic axial compression</td>
</tr>
<tr>
<td>P06</td>
<td>Quasi-static cyclic axial tension-compression</td>
</tr>
<tr>
<td>P07 to P10</td>
<td>Quasi-static cyclic axial tension-compression</td>
</tr>
<tr>
<td>P11 to P17</td>
<td>Quasi-static cyclic axial tension-compression</td>
</tr>
</tbody>
</table>

#### 5.2.3.1 Quasi-Static Monotonic Axial Compression Loading Protocol

Test specimens P01 and P05 had a quasi-static monotonic axial compression loading protocol applied to them. The specimens were in axial compression under a displacement-controlled protocol until failure.

#### 5.2.3.2 Quasi-Static Cyclic Axial Tension-Compression Loading Protocol

Test specimens P02, P03, P06 and P07 to P17 had a quasi-static cyclic axial tension-compression loading protocol applied to them. Each respective specimen was subjected to a preselected tension displacement increment in displacement-controlled behaviour and then the load was reversed and axial compression was applied to the specimen until the cracks closed (i.e. axial stiffness was regained) or the capacity of the machine was reached (i.e. 1 MN). If the specimen was able to recover its axial stiffness, i.e. the cracks closed without any localised damage occurring, the specimen was then subjected to larger tension displacement and the process repeated.
The failure mechanisms of interest in this testing, i.e. global out-of-plane buckling and local bar buckling, occur after the element is subjected to reversed axial compression prior to the cracks closing. If the cracks close without any localised damage occurring, the wall can regain its axial stiffness and these failure mechanisms are not able to occur and the compression loading could be paused and the subsequent increased tension displacement increment could be applied (refer to Section 4.3.4.2 and 4.3.4.3 respectively for a further discussion regarding these failure mechanisms).

The specimens were typically loaded to tension displacement increments that equalled 1%, 2%, 3%, 4% and 5% average strain across the height of the specimen (e.g. 1% average strain in P01 equals 2000 × 0.01 = 20 mm), unless failure occurred prior. Additionally, test specimens P07 to P17 were initially loaded to a tension force that equalled a stress between 350 to 450 MPa (i.e. 70% to 90% of the characteristic yield stress) being reached in the vertical reinforcement, before starting the tension displacement increments stated above. This was done to assess residual crack widths when the end region of wall is subjected to pre-yield tension strains. An illustrative loading protocol for all the specimens is presented below in Figure 5.7.

![Illustrative loading protocol for the cyclic axial tension-compression specimens.](image)

**Figure 5.7:** Illustrative loading protocol for the cyclic axial tension-compression specimens.

### 5.2.3.3 Quasi-Dynamic Cyclic Axial Tension-Compression Loading Protocol

Test specimen P04 was tested using a quasi-dynamic cyclic axial tension-compression loading protocol. The loading protocol was much the same as the quasi-static loading protocol described above (i.e. Figure 5.7), however the loading rate was significantly faster and intended to represent indicative strain rates that would be typical of the response seen during an earthquake.

The specimen was initially loaded at a velocity of 140 mm/s (i.e. a strain rate of 7% per second), however the test machine was unable to apply the loading at this rate, as shown by the commanded and actual displacement signals in Figure 5.8. Therefore, the test was paused at approximately the 2 second mark and the loading rate was adjusted to 100 mm/s (i.e. a strain rate of 5% per second). The test machine however, was still unable to achieve the slower loading rate of 100 mm/s and the actual loading rate for the third and fourth loading cycles was approximately 50 and 70 mm/s respectively (i.e. a strain rate of 2.5% and 3.5% per second respectively). Strain rates of 2.5% to 3.5% would still be considered representative of the response expected during an earthquake [P1, S14].
5.3 Test Results

The results of the boundary element prism testing are presented and discussed in two subsequent sub-sections. Section 5.3.1 presents the type 1 specimens results (i.e. the high slenderness ratio specimens) and Section 5.3.2 presents the type 2 specimen results (i.e. the low slenderness ratio specimens). The conclusions and final remarks from both sets of specimens is then presented in Section 5.3.3.

5.3.1 Type 1 Specimens – P01 to P06

5.3.1.1 Quasi-Static Monotonic Axial Compression Tests – P01 and P05

Test specimens P01 and P05 failed in combined compression and eccentric bending, which resulting in a diagonal failure plane developing at the base of the wall, across the thickness of the wall, as shown in Figures 5.9(a) and 5.9(e) respectively. Despite best efforts to locate the specimen in the centre of the machine and apply concentric loading, it is believed the walls were loaded with a slight horizontal eccentricity, resulting in the combined compression and bending behaviour.

Figure 5.9: Failure photos of the type 1 boundary element test specimens.
Specimens P01 and P05 failed at a total vertical load of 2040 and 1954 kN respectively. The failure loads corresponded to 78% and 62% respectively of the theoretical maximum capacity of the concrete (i.e. the gross cross-sectional area multiplied by the concrete strength given in Table 5.3). The average strain, taken across the full height of the specimen, at failure was 0.18% and 0.15% respectively. The force-displacement response of both specimens is presented in Figure 5.10.

![Force-displacement response of test specimen P01 (right) and P05 (left).](image)

**Figure 5.10:** Force-displacement response of test specimen P01 (right) and P05 (left).

The amount of accidental horizontal eccentricity that was applied to each test specimen was determined by calculating the maximum moment capacity of the section, which corresponds to the maximum applied vertical load of the respective specimen discussed above. The maximum moment capacity was calculated using the fibre-element analysis method described in Section 4.3.2 and the AS 3600 Supp1 [X36] stress-strain relationship for concrete, which is presented in Section 4.4.1. The maximum moment capacity of specimens P01 and P05 was calculated to be 28.4 and 40.9 kNm. The accidental horizontal eccentricity can then be calculated by divided the maximum moment capacity by maximum applied vertical load, i.e. \( e = \frac{M^*}{N^*} \). This results in an eccentricity of 14 and 21 mm respectively for specimens P01 and P05. The stress and strain diagrams from the fibre-element analysis for P01 and P05 are presented below in Figure 5.11.

![Fibre-element analysis stress and strain diagrams for test specimens P01 and P05.](image)

**Figure 5.11:** Fibre-element analysis stress and strain diagrams for test specimens P01 and P05.
The ultimate compression capacity of each wall was calculated using the procedure proposed by AS 3600 [X6], i.e. Equation 5.2. The horizontal eccentricity values calculated above were used in Equation 5.2, in addition to the concrete strengths presented in Table 5.3. Further, a capacity reduction factor of 1.0 was also assumed, however it should be noted that AS 3600 requires a capacity reduction factor of 0.6 for RC walls in compression. This procedure resulted in an ultimate compression capacity of 1069 and 1158 kN respectively for specimens P01 and P05, which is equal to a mere 52% and 59% of the maximum vertical load each specimen was able to withstand. A summary of all the results for specimens P01 and P05 is presented in Table 5.12.

\[
\phi N_u = \phi(t_w - 1.2e - 2e_a)0.6f'_c
\]

Where:
- \( N_u \) = ultimate strength per unit length of wall
- \( \phi \) = capacity reduction factor
- \( f'_c \) = concrete characteristic compressive strength
- \( t_w \) = thickness of the wall
- \( e \) = eccentricity of load (refer Clause 11.5.2 of AS 3600)
- \( e_a \) = additional eccentricity taken as \( H_e^2/(2500t_w) \)
- \( H_e \) = effective height of the wall

<table>
<thead>
<tr>
<th>Table 5.12: Summary of results for test specimens P01 and P05.</th>
<th>P01</th>
<th>P05</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>( N_{\text{max}} = )</td>
<td>2040 kN</td>
<td>1954 kN</td>
<td>Maximum compressive strength of the test specimen</td>
</tr>
<tr>
<td>( \epsilon_{\text{ave}} = )</td>
<td>0.18%</td>
<td>0.15%</td>
<td>Average strain corresponding to the maximum compressive strength being reached</td>
</tr>
<tr>
<td>( M_{\text{max}} = )</td>
<td>28.4 kNm</td>
<td>40.9 kNm</td>
<td>Maximum moment capacity of the cross-section corresponding to ( N_{\text{max}} )</td>
</tr>
<tr>
<td>( e = )</td>
<td>14 mm</td>
<td>21 mm</td>
<td>Horizontal eccentricity, i.e. ( e = M_{\text{max}}/N_{\text{max}} )</td>
</tr>
<tr>
<td>( N_u = )</td>
<td>1069 kN</td>
<td>1158 kN</td>
<td>Ultimate compression capacity in accordance with AS 3600</td>
</tr>
<tr>
<td>( N_{\text{gross}} = )</td>
<td>2615 kN</td>
<td>3130 kN</td>
<td>Theoretical gross compression capacity of the cross-section (i.e. gross cross-section area multiplied by maximum concrete strength)</td>
</tr>
<tr>
<td>( N_{\text{max}}/N_u = )</td>
<td>1.91</td>
<td>1.69</td>
<td>Ratio of the actual compression capacity of the wall to code strength</td>
</tr>
<tr>
<td>( N_{\text{max}}/N_{\text{gross}} = )</td>
<td>0.78</td>
<td>0.62</td>
<td>Ratio of the actual compression capacity of the wall to gross strength</td>
</tr>
</tbody>
</table>
5.3.1.2 Quasi-Static Cyclic Axial Tension-Compression Tests – P02, P03 and P06

Test specimens P02 and P03 failed in the desired out-of-plane buckling failure mechanism, as shown in Figures 5.9(b) and 5.9(c) respectively. The specimens were able to undergo 2.0% and 1.8% average tension strain respectively, without an out-of-plane buckling failure occurring in the subsequent reversed axial compression load cycle. When the specimens were then subjected to larger average tension strains of 3.0% and 2.8% respectively, out-of-plane buckling occurred in the subsequent reversed axial compression load cycle. The average tension strain is equal to the tension displacement divided by the height of the wall, i.e. 40 mm displacement equals 2% average tension strain (40/2000 = 0.02). The out-of-plane buckling behaviour and results of specimens P02 and P03 are presented in Figures 5.12 and 5.13 respectively. The normalised force in these figures is equal to the force divided by the gross cross-sectional area of wall multiplied by its concrete strength given in Table 5.3.

The observed out-of-plane buckling behaviour is summarised as follows:

1. The RC element is subjected to an axial tension displacement that cracks the concrete and forces the tension load to be transferred to the vertical reinforcement. The reinforcement then yields and resists the axial tension displacement through a combination of elastic elongation and inelastic plastic elongation.
2. The tension load on the RC element is released and the in-plane displacement is reduced by the elastic portion of elongation in the reinforcement being recovered. At this point there are residual cracks ‘still open’ due to the inelastic plastic strains in the reinforcement from the previous tension displacement load cycle.
3. The RC element is subjected to compression. The compression load allows the inelastic plastic strains to be recovered and initiates closing the residual crack width. As the residual cracks begin to close, out-of-plane displacement of the wall initiates and progressively increases as the cracks start to close.
4. A bifurcation effect occurs, where depending on the amount of inelastic plastic tension strain in the reinforcement and the overall slenderness ratio of the wall, either:
   i. The wall ‘self-corrects’ and the out-of-plane displacement reduces back to zero as the cracks begin to close and the axial stiffness of the wall is regained.
   ii. The wall cannot self-correct, the axial stiffness of the wall cannot be regained and out-of-plane buckling failure occurs.

This behaviour is shown visually in Section 5.3.1.3 for test specimen P04. Stages 1, 2, 3 and 4i are shown in Figures 5.18(a), 5.18(b), 5.18(c) and 5.18(d) respectively and stages 1, 2, 3 and 4ii are shown in Figure 5.18(e), 5.18(f), 5.18(g) and 5.18(h) respectively.

When out-of-plane buckling failure occurred, the compression load on the walls was 285 and 288 kN for specimens P02 and P03 respectively, which is equal to 11% and 9% respectively of the theoretical gross strength of the wall. Further, for specimen P02, 285 kN is equal to 14% of the maximum compressive load of the matching monotonically loaded specimen P01.

The local strain in the reinforcement for specimens P02 and P03 was indirectly calculated using the tension-stiffening model developed later in Section 5.6. For test specimen P02 the local strains in the reinforcement corresponding to the average tension strain values of 1.0%, 2.0% and 3.0% were 3.0%, 3.7% and 4.3% respectively. Similar for test specimen P03, the local strains for the average tension strain values of 0.8%, 1.8% and 2.8% were 2.6%, 3.0% and 3.3% respectively.
Figure 5.12: Test specimen P02 response.

Figure 5.13: Test specimen P03 response.

Figure 5.14: Test specimen P06 response.
Test results for specimen P02 and P03 were compared to the models proposed by Paulay and Priestley [P11] and Chai and Elayer [C8], presented in Section 4.3.4.2, which can be used to calculate the maximum tension strain a wall can be subjected to prior to out-of-plane buckling occurring in the subsequent reversed load cycle. This comparison is presented in Figure 5.15. While these proposed models are for calculating the maximum tensile strain in the reinforcement, the Chai and Elayer [C8] model, which was developed using a comprehensive study consisting of 14 test specimens, was developed and calibrated using the average tension strain values from the test specimens, not the actual local tension strain in the reinforcement. This means the comparison in Figure 5.15 uses the average tension strain values from P02 and P03 (i.e. 1.0%, 2.0% and 3.0% for P02 and 0.8%, 1.8% and 2.8% for P03). Further, the test specimens in Chai and Elayer [C8] were tested with pin ended boundary restraints, whereas it could be argue that test specimens P02 and P03 had a fixed or semi-fixed boundary restraint. Paulay and Priestley [P11] also argue that when calculating the slenderness ratio, it should be the minimum of the plastic hinge length divided by the thickness of the wall or 80% of the clear story height divided by the thickness of the wall. Therefore, when comparing the test results in Figure 5.15, the slenderness ratio of specimens P02 and P03 was taken to be 12.3, i.e. $0.8 \times \frac{2000}{130} = 12.3$.

![Comparison of out-of-plane buckling results to models in literature.](image)

It can be seen in Figure 5.15 that both the Paulay and Priestley [P11] and Chai and Elayer [C8] models are somewhat conservative for predicting maximum tensile strains prior to buckling occurring on subsequent reversed axial compression load cycles. Further, both models give maximum strains that correspond to the average tension strain of the section, not the local tensile strain in the reinforcement. The difference between these values can vary significantly and is largely dependent on the crack spacing. When undertaking many forms of structural analysis for RC sections, e.g. fibre-element analysis, the concrete is often assumed to have zero tensile capacity and as such, the tensile strains from the analysis are actually the local tensile strains in the reinforcement, not the average tensile strain of the section. This means care should be taken when adopting either of these proposed models to calculate tensile strain limits for an analysis model, particular fibre-element analysis models, as it would likely result in a very conservative lower bound strain value, meaning the capacity of the element could be significantly under estimated.
Test specimen P06, unlike specimen P02 and P03, failed due to local bar buckling of the vertical reinforcement, prior to an out-of-plane buckling failure could occur. Specimen P06 behaved similar to specimens P02 and P03 during the first loading cycle (as shown in Figure 5.14). However, during load cycle two, after the load reversal from axial tension to axial compression and the wall regained its initial axial stiffness, local bar buckling of the vertical reinforcement occurred. The behaviour of specimen P06 during load cycle two, up to failure, is shown in Figure 5.16. It should be noted that the initial loading cycle for specimen P06 was interrupted due to a system error on the test machine. This did not affect the behaviour or response of the specimen, however it did result in the first portion of test data not being captured, as indicated by the dashed line in Figure 5.14.

The behaviour of specimen P06 was attributed to the irregular and sparse distribution of cracking, which was due to the low percentage of vertical reinforcement. This resulted in the formation of one large crack just below the mid-height of the wall (refer Figure 5.16(a)). During load cycle 2, the maximum axial tension displacement was 40 mm and the one major crack had a crack width of approximately 12 mm, meaning 30% of the axial tension displacement was concentrated at one location. The large concentration of displacement across this single crack would have meant a very large local strain in the reinforcement at this location. A combination of this larger amount of local strain and the lack of distributed cracking (unlike the well distributed closely spaced cracking that was observed in specimens P02 and P03), was believed to have initiated the local bar buckling behaviour of the vertical reinforcement.

![Test specimen P06 failure progression during load cycle two.](image)

**Figure 5.16:** Test specimen P06 failure progression during load cycle two.

### 5.3.1.3 Quasi-Dynamic Axial Tension-Compression Tests – P04

The results of the quasi-dynamic axial tension-compression test specimen (i.e. P04), which was a replica of specimen P03, are presented in Figure 5.17. The out-of-plane buckling behaviour of RC walls is depicted by a series of photos taken during the testing of specimen P04, which are presented in Figure 5.18. Reference markers for each photo are show on the response curves in Figure 5.17. The results of the quasi-dynamic test (i.e. P04) are compared to the results of quasi-static test (i.e. P03) in Figure 5.19. This comparison indicates that strain rate has little effect on the out-of-plane buckling behaviour.
Figure 5.17: Test specimen P04 response.

(a) point 1  (b) point 2  (c) point 3  (d) point 4

(e) point 5  (f) point 6  (g) point 7  (h) point 8

Figure 5.18: Test specimen P04 out-of-plane buckling mechanism.
5.3.2 Type 2 Specimens – P07 to P17

Failure photos for the round one test specimens (i.e. P07 to P10) are presented in Figures 5.20(a) to 5.20(d) respectively. Failure photos for the round two test specimens (i.e. P11 to P17) are presented in Figures 5.21(a) to 5.21(g) respectively.

The force-displacement response of test specimens P07 to P17 is presented in Figures 5.22 and 5.23. The response is presented in terms of the normalised force and average global strain. The former is equal to the force divided by the tensile yield capacity of the specimen (i.e. the force divided by the product of the total cross-sectional area of vertical reinforcement and the respective yield stress of the bars given in either Table 5.7 or 5.1) and the latter is equal to the axial displacement divided by the height of the specimen (i.e. $\Delta H_w$). For test specimens P15, P16 and P17 where the cross-sectional area of vertical reinforcement and dowel bars was not equal, the normalised force is presented with respective to the lesser of the two. This means the normalised force for test specimen P15 is with respect to the tensile yield capacity of the dowel bars and for test specimens P16 and P17 it is with respect to the tensile yield capacity of the vertical reinforcement.

5.3.2.1 Failure Mechanism of Round One Test Specimens – P07 to P10

Test specimens P07 and P10 failed via local bar buckling, as shown in Figures 5.20(a) and 5.20(d) respectively. Bar buckling occurred in specimen P07 in the reversed axial compression cyclic after being subjected to a 40 mm axial tension displacement, which corresponds to an average tension strain of 5%. The strain gauges on the vertical bars ‘saturated’ after the previous cycle, so the local tension strain corresponding to the 5% average tension strain load increment was unknown. However, the previous load cycle (i.e. average tension strain of 4%) the local tension strain was 6.2%. Further, using the model develop and calibrated latter in Section 5.6, the local tension strain corresponding to the average tension strain of 5%, was calculated to be approximately 7.3%. This means that the vertical reinforcement could be subjected to a local tension strain of about 6–7% before local bar buckling was initiated.

Bar buckling occurred in specimen P10 in the reversed axial compression cycle after being subjected to a 23 mm axial tension displacement, which corresponds to an average tension strain of 2.8%. The local tension strain at this point was 4.5%.
Figure 5.20: Failure photos of the type 2 round 1 boundary element test specimens.

Figure 5.21: Failure photos of the type 2 round 2 boundary element test specimens.
Figure 5.22: Test specimens P07 to P12 force-displacement response.
Figure 5.23: Test specimens P13 to P17 force-displacement response.
It is believed that vertical splitting of the specimen, which can be seen Figure 5.20(d), resulted in vertical bar buckling being initiated at a lower tension strain than specimen P07. This indicates that local bar buckling is potentially worse in singularly reinforced walls, since in doubly reinforced walls there would normally be some nominal cross ties (e.g. 'U' bars are each end of the wall, refer Figure 3.8(c)), that would prevent vertical splitting from occurring.

Test specimen P08 failed via global out-of-plane buckling, as shown in Figure 5.20(b). The out-of-plane buckling mechanism matched what was observed in specimens P02, P03 and P04, as shown in Figures 5.9(a), 5.9(b) and 5.9(c) respectively and discussed in Section 5.3.1. The global buckling occurred after a tension displacement of 37 mm was applied, which corresponded to an average tension strain of 4.7% and a local strain in the reinforcement of 4.8%. The results of specimens P07 and P08 show that the vertical reinforcement in an RC wall can be subjected to a local tension strains of 5% without local bar buckling occurring in subsequent reversed load cycles.

The out-of-plane buckling behaviour, expressed in terms of out-of-plane displacement verse axial displacement, of specimen P08 is shown in Figure 5.24. These results match very closely to what is shown in Figures 5.12 and 5.13 for test specimens P02 and P03 respectively. The crack propagator in specimen P08 meant that the thickness of the wall at mid-height was reduced from 130 mm to 100 mm (refer Figure 5.3), which allowed buckling to occur more easily. This means it would likely be more appropriate to conservatively take the slenderness ratio (i.e. $H_w/t_w$) of specimen P08 as $800/100 = 8.0$, instead of the value of $800/130 = 6.2$ given in Table 5.5.

![Figure 5.24: Out-of-plane buckling response of test specimen P08 (left) and comparison between specimens P03 and P04 (right).](image)

Test specimen P09 failed via bar fracturing of the vertical reinforcement, as shown in Figure 5.20(c). The specimen was able to be subjected to a tension displacement of 33 mm, which corresponding to an average tension of 4.1%, before the fracturing of the vertical reinforcement was initiated on the following tension displacement load cycle at a tension displacement of approximately 29 mm or 3.6% average tension strain.

The maximum tension forces, equivalent average reinforcement stresses, tension displacements, average tension strains and local tension strains, which were determined from the respective strain gauges, for each tension load cycle are tabulated in Appendix B.
5.3.2.2 Failure Mechanism of Round Two Test Specimens – P11 to P17

Test specimens P11 and P13 failed via local bar buckling, as shown in Figures 5.21(a) and 5.21(c) respectively. Bar buckling occurred in both specimens in the subsequent reversed axial compression load cycle after being subjected to tension displacements of 31 and 33 mm respectively, which correspond to average tension strains of 3.8% and 3.9% respectively.

The specimens are subjected to axial tension with a constant strain gradient, meaning that at each crack location, the axial force is purely resisted by the vertical reinforcement. Therefore, the average local tension strain in the reinforcement, at each crack, can be estimated using an average reinforcement stress-stain curve for each specimen, which was calculated from tensile test of reinforcement material samples. The average stress-strain curves for the reinforcement in each specimen are presented in Appendix B.

Using this approach, the average local tension strain in the reinforcement prior to bar buckling occurring in specimen P11 was calculated to be approximately 5.4%. Further, it was observed during the test that the specimen did not appear to have a constant strain distribution across the length of the specimen, meaning the crack widths were not constant across the length of the specimen. This is shown using the results from the photogrammetry system in Figure 5.25, which shows the average strain distribution at the front of the wall and the rear of the wall. It can be seen here that twist of the specimen occurred where at the 300 mm mark the rear of the wall had higher strains the front and then at the 500 mm mark, the reverse occurred. The bar buckling occurred at the 500 mm mark, where the strains at the front were approximately 70% higher than the strains at the rear of the specimen. Further, the strain at the front of the wall was 26% greater than the average of the strain of the front and rear combined. Therefore, the average local tension strain calculated above should be proportionately increased by 26% to take this into account. This results in the local tension strain being approximately 6.8% prior to bar buckling occurring.

Test specimen P13 had a similar crack distribution to P11, failed after being subjected to approximately the same loading increment and had very similar material properties (refer Tables 5.9 and 5.1). Therefore, it would be appropriate to assume that the specimen had a similar local tension strain to P13 prior to failure.

![Figure 5.25: Test specimen P11 strain distribution (tension displacement of 31 mm).](image)
Test specimens P12 and P14 failed via bond failure of the lap splice (i.e. unzipping), as shown in Figures 5.21(b) and 5.21(d) respectively. This is believed to be the result of two factors. The first and primary factor being that specimens P12 and P14 had a significantly lower concrete strength than what was specified (as discussed in Section 5.1.2). The second factor was also due to the orientation in which the specimens were poured. Test specimens P01 and P10 were poured such that the 450 mm length was ‘flat’, where specimens P11 to P17 were poured such that the 450 mm length was ‘vertical’, as shown in Figures 5.26(a) and 5.26(b) respectively. When horizontal bars are placed in RC elements where there is greater than 250 to 300 mm of concrete poured below them, there can be a “reduction in bond strength due to settlement of the fresh concrete and an accumulation of bleed water along the underside of the bar” [X36]. It is believed however, if the concrete strength was not 20% lower than what was specified, the debonding would not have occurred.

![Formwork arrangement for P01 to P10](image1)

![Formwork arrangement for P11 to P17](image2)

**Figure 5.26:** Formwork arrangement for test specimens P01 to P17.

Test specimens P15, P16 and P17 failed via bar fracturing of the vertical reinforcement, as shown in Figures 5.21(e), 5.21(f) and 5.21(g) respectively. Specimen P15 was detailed such that the dowel connection was the ‘weak’ point. This was done by using 3-N16 dowel bars and then 8-N12 vertical bars in the wall section above, meaning the vertical reinforcement in the wall above had 1.5 times the capacity of the dowel bars in tensions. As a result, the majority of the displacement was concentrated at the base and only hairline cracks developed across the height of the specimen. The wall was able to develop an average tension stress of 673 MPa before bar fracture occurred, which was 3% less than the ultimate stress of the bars, indicating that the tension load was not evenly distributed amongst the three dowel bars.

Specimen P15 was able to resist 406 kN in tension prior to the first bar fracture occurring. This means that the 40 mm diameter corrugated grout tubes, which were 500 mm long, were able to develop an average bond stress of 2.2 MPa. This corresponds to a bond stress of approximately \(0.4\sqrt{f_{cm}}\), where \(f_{cm}\) is the average compressive strength of P15 given in Table 5.9. The maximum bond stress of the corrugated grout tubes would likely be significantly higher than this.

Specimen P16 was detailed such that the section of wall above the dowel connection was the ‘weak’ point. This was done using 4-N20 dowel bars and 6-N12 vertical bars in the wall section above, meaning the dowel bars had 1.9 times the capacity of the vertical bars in the wall section. As a result, the majority of the displacement was concentrated in the 350 mm section of wall above the grout tube connection and only hairline cracks developed across the grout tube region of the specimen.
The inelastic tension displacement in specimen P16 was concentrated across three major cracks. This meant the specimen failed at a much small tension displacement increment than P11 and P13. Interestingly enough, despite the development of a major crack, with what would have had a significantly high amount local tension strain in the reinforcement, unlike specimens P07, P10, P11 and P13, bar buckling was not initiated. Similar to P15, the wall was able to developed an average tension stress of 619 MPa, which was 5% less than the ultimate stress of the bars. Meaning the tension load was not evenly distributed amongst the six vertical bars.

It is difficult to ascertain from the results of P15 and P16 as to whether better performance will be achieved from a precast RC wall if it is designed to have the ‘weak’ point above the grout tube or at the base of the wall. If the latter occurs, the inelastic behaviour will be concentrated at the base of the wall and a ‘singe-crack’ mechanism will be formed (i.e. Figure 3.14(b)). This type of mechanism in cast in-situ walls has been heavily researched and found to result in quite poor seismic performance, as discussed in detail in Section 3.5.4. However, in precast walls with a grout tube connection, it seems that the grout has poorer inelastic bond strength properties compared to in-situ concrete and as such, much greater yield penetration occurs prior to bar fracturing, resulting in significantly more inelastic behaviour or a ‘wider’ crack at the base of the wall. It was also observed in P15 and building core specimens S03 and S04, which are presented later in Chapter 7, that the foundation block and grout tube prevent bar buckling of the dowel bar, allowing tensile fracturing of the dowel bar to be the ultimate failure mechanism. Alternatively, if the weak point is above the grout tube, a typical plastic hinge could be developed (i.e. Figure 3.14(a)), which has traditionally been the desirable failure mechanism for walls. Further analytical studies are recommended in this instance before design advice is provided.

Specimen P17 was detailed to generally match P16, except the vertical bars were substituted for low ductility reinforcement (i.e. D500L to AS/NZS 4671). Low ductility reinforcement has a very small minimum characteristic strain hardening ratio (i.e. $f_{su}/f_{sy}$) and typically, the little strain hardening it can sustain, occurs over very short region from the onset of yielding to about 1% strain being reached (refer Figure 3.16). Between 1% strain and the uniform elongation of the bar, typically only a minor amount of additional strain hardening occurs. This extremely inhibits a member’s ability to develop distributed cracking with distributed plasticity. This was observed experimentally in specimen P17, where despite three main cracks developing between the top of the grout tube connection and the top of the specimen (as shown later in Figure 5.38), once the vertical bars yielded, all the inelastic deformation was concentrated in one crack and resulted in bar fracture of the vertical reinforcement at a very small tension displacement.

### 5.3.3 Summary of Results

The key results from the boundary element prism testing are as follows:

1. Global out-of-plane buckling of occurs when an RC element is in compression and tries to overcome the residual inelastic plastic tension strains sustained in the previous reserved cyclic load case. The wall undergoes a bifurcation effect where it either overcomes the residual plastic tension strains and regains its initial axial stiffness or it cannot overcome the residual plastic tension strains and out-of-plane buckling occurs, which results in axial load failure of the element. The axial load failure occurs at a load equal to about 10% of the gross compression capacity of the element.
2. Strain rate seems to have little effect on the out-of-plane buckling mechanism of RC walls, meaning traditional pseudo-static loading protocols are appropriate for experimentally assessing this behaviour and failure mode of wall.
3. Bar buckling in limited ductile RC construction is a complex failure mechanism as it relies on the cover concrete to provide lateral restraint against buckling. The buckling force is dependent on both the residual amount of inelastic tension strain in the bar and the residual crack width. It has been shown empirically in this testing that reinforcing bars of various diameters – including 10, 12 and 16 mm nominal diameter bars – can undergo local tension strains in the reinforcement of about 6–7% before bar buckling is initiated in the subsequent reserved load cycle.

4. Walls detailed with a single central layer of vertical and horizontal reinforcement perform significantly worse under reversed cyclic loading than walls that are reinforced with a layer of vertical and horizontal reinforcement on each face. It was shown experimentally that they undergo significantly higher amounts of out-of-plane displacement for the relative equivalent tension strain in the previous reversed load cycle. Further, it was also shown that bar buckling is initiated after much smaller tension strain values.

5. Low ductility reinforcement (i.e. D500L mesh) extremely inhibits an elements ability to develop distributed cracking with distributed plasticity due to the limited amount of strain hardening the bar is able to undergo. It was shown experimentally that the low strain hardening ratio of the bars result in all the plastic deformations being concentrated in one crack, which then coupled with the low ultimate strain of the bars, leads to bar fracture at relative low displacement increments compared to normal ductility reinforcement.

5.4 Cyclic Behaviour of Reinforcement

A series of monotonic and cyclic tensile tests were performed to better understand the cyclic behaviour of reinforcement. The test samples were 24 mm nominal diameter reinforcing bars, which were all cut from the same length of rebar. The reinforcing bar was a deformed normal ductility bar with a minimum characteristic yield stress of 500 MPa, i.e. D500N to AS/NZS 4671.

Three different loading protocols were adopted and three samples were tested for each loading protocol. The first loading protocol (i.e. Figure 5.27) applied a cyclic tension load in approximately 2% strain increments. The load was released back to zero force after each strain increment. The second loading protocol (i.e. Figure 5.28) also applied a cyclic tension load, however after each increment when the load was released, a compression force equal to 50% of the yield stress of the bar was applied. The third loading protocol (i.e. Figure 5.29) was similar to the second loading protocol, however instead of applied a compression load equal to 50% of the yield stress of the bar, a compression load until bar buckling occurred was applied. Additionally, three monotonic tests were also completed to determine the ‘baseline’ properties of the bar. The average monotonic stress-strain response was determined by averaging each individual stress-strain curve together. The average monotonic response is overlaid on each cyclic response curve in Figures 5.27 to 5.29. The stress axes in Figure 5.29 have been normalised by divided the stress by the monotonic yield stress.

The first two sets of loading protocol specimens (i.e. Figures 5.29 and 5.28) show that the cyclic response of the reinforcement is approximately equal to the backbone monotonic response. However, with the third loading protocol set of specimens (i.e. Figure 5.29) the response somewhat varies from the backbone monotonic behaviour. The bars follow the same initial response, i.e. elastic response, yield plateau and initial portion of strain hardening, however the maximum tensile stress is reached at a strain equal to about 60% of the uniform elongation (i.e. ultimate strain) of the monotonic behaviour. This shows that cyclic loading does not affect or reduce the behaviour of the bar unless the bar buckles in the previous reversed compression cycle.
This section will propose a set of material strain limits for the displacement-based design of limited ductile RC walls. The strain limits being proposed are generally in accordance with the limits proposed by [P4, S3], which are summarised in Section 4.3.5 and Table 4.3. The limits are proposed with respect to the same categories proposed by these authors, namely ‘no damage’, ‘repairable damage’ and ‘no collapse’. With respect to the Australian design context (i.e. AS/NZS 1170.0 [X22]), it is being proposed the no damage performance criteria aligns to serviceability limit state (SLS) and the no collapse performance criteria aligns to ultimate limit state (ULS). The discussion and justification for this assertion is provided in Section 2.2. The proposed material strain limits are summarised in Table 5.13 and discussed further in the following sub-sections.
Table 5.13: Proposed material strain limits for limited ductile construction.

<table>
<thead>
<tr>
<th>Material strain limit</th>
<th>No damage (i.e. SLS)</th>
<th>Repairable damage</th>
<th>No collapse (i.e. ULS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strain, $\varepsilon_c$</td>
<td>0.2%</td>
<td>0.4%</td>
<td>0.6%</td>
</tr>
<tr>
<td>Reinforcement tension strain, $\varepsilon_s$</td>
<td>0.5%</td>
<td>1.5%</td>
<td>5.0%</td>
</tr>
</tbody>
</table>

5.5.1 Concrete Compressive Strain Limits

Concrete compressive strain limits of 0.2%, 0.4% and 0.6% are being proposed for no damage, repairable damage and no collapse performance objectives respectively. The strict 0.2% limit is being proposed for the no damage performance objective as this will ensure essentially linear elastic response of the structure. In the Australian design context, the no damage (i.e. serviceability limit state) performance objective is required to be met for importance level 4 buildings in accordance with AS 1170.4 [X4]. Wherein it requires importance level 4 buildings remain operational following the equivalent ultimate limit state event for an importance level 2 building. While the full requirements to ensure an importance level 4 building remains operation following an earthquake will not be discussed within this thesis, an example would include ensuring that the RC walls surrounding the lift shafts do not undergo excessive compression and or tension forces such that wide cracks or spalling occurs, as this may compromise the integrity of the fixings between the lift rails and the RC walls and hence, effect the operational capacity of the lift shafts. This would be of particular importance in hospitals that have operating theatres located on any other floor besides ground.

The concrete compressive strain limits of 0.4% and 0.6% for the repairable damage and no collapse performance objectives respectively are being proposed based on the recommendations by Sullivan et al. [S3]. These two values are essentially the limits they propose for unconfined concrete. Confinement is rarely provided in limited ductile walls, as reported in Chapter 3. Further, the concrete compressive strain limits have to be such that they prevent spalling from occurring, as once the cover concrete is lost in limited ductile RC elements, bar buckling will ensure immediately given the lack of confinement. Typically, a value of 0.3% is proposed as the ultimate compressive strain of concrete, however as pointed out by Priestley et al. [P4], the "maximum compressive strains almost always occur adjacent to a supporting member (e.g. a foundation beam for a concrete column or wall), which provide an additional restraint against initiation of spalling”. This usually means a higher concrete compressive strain can be achieved prior to cover spalling being initiated. This was shown experimental in the large-scale cast in-situ RC wall testing in Chapter 6, where both walls were able to achieve compressive strains greater than 0.6% at the base of the wall (i.e. adjacent to the foundation block) without bar buckling.

5.5.2 Reinforcement Tension Strain Limits

Reinforcement tensile strain limits of 0.5%, 1.5% and 5% are being proposed for no damage, repairable damage and no collapse performance objects respectively. The strict 0.5% limit, which equals a strain approximately two times the yield strain of typical grade D500N reinforcement, is being proposed for the no damage performance objective as this will ensure essentially linear elastic response of the structure. The same justification for linear elastic response provided in the previous section applies here again.
A reinforcement tensile strain limit of 1.5% for the repairable damage performance objective is being proposed based on reports by Priestley et al. [P4, P18] that "for structural elements with compression gravity loading (walls, columns), a maximum tension strain of 0.015 during seismic response will correspond to residual crack widths of about 1.0 mm". It is also noted that Priestley et al. recommend a smaller tension strain limit of 0.010 for elements without axial compression (e.g. beams) to ensure the residual crack widths are limited to 1.0 mm.

A reinforcement tensile strain limit of 5.0% for the no collapse performance objective is being proposed to ensure local bar buckling of the reinforcement does not occur. It was shown experimentally in this chapter that an RC element could sustain local tension strains in the vertical reinforcement of about 6–7% before being susceptible to bar buckling on reversed load cycles. Further, AS/NZS 4671 requires a minimum characteristic uniform elongation (i.e. ultimate tensile strain) of 5.0% for D500N reinforcement, which is the standard grade of reinforcement available in the Australian market. However, it’s noted in practice though, the ultimate strain is typically much greater than 5.0% and is usually around 8–9% (refer Section 4.4.2). The 5.0% limit conveniently satisfies both these requirements.

Hilson [H10] performed a series of prism tests in the U.S., which had similar reinforcement detailing to what would usually be seen in limited ductile walls. These specimens underwent bar buckling after being subjected to much lower tension strains than what was observed in the specimens in this chapter; bar buckling was observed after tension strains of about 3–4.5%. The tension strains reported in this work however, were average tension strains over a gauge length, as opposed to local tension strains in the reinforcement. This is the likely reason for the significant difference between the values reported by Hilson [H10] and the values reported in this chapter. The recommended tension strain limits in this section are intended to be local reinforcement tension strain limits.

It should be noted that these limits do not provide any consideration for avoiding out-of-plane buckling failures. This is a complex failure mechanism that is dependent on the local inelastic tension strain in the reinforcement, the crack width and the slenderness ratio of the wall. Further research is required before firm guidance – with respect to tension strain limits – to avoid this failure mechanism can be proposed. In the interim, Wallace [W12] has recommended limiting the slenderness ratio to 16 or less to avoid this mode of failure. A slenderness ratio of 16 or less would only need to be applied in plastic hinge regions or elsewhere in RC walls where significant elastic plastic tensions strain could be developed.

### 5.6 Tension Stiffening in Limited Ductile RC Walls

RC members subjected to uniaxial tension will initially resist the applied load by developing a combination of elastic tensile stress in the concrete and reinforcement. Cracking of the concrete will then initiate after the maximum tensile stress of the concrete has been exceeded and result in the reinforcement providing the sole tensile resistance wherever the concrete has cracked. In between adjacent cracks the reinforcement will transfer a portion of the tensile load back into the concrete due to the mechanical interlock between the reinforcement and concrete (i.e. bond stress). This mechanism, which is referred to as tension stiffening, means the tensile stiffness of the element is not equal to the tensile stiffness of the reinforcement, rather a value somewhat higher. The actual tensile stiffness depends on the crack spacing, which is dependent on the maximum tensile strength of the concrete, the bar diameter of the vertical reinforcement and the bond strength.
The tension stiffening behaviour essentially results in there being a difference between the local tension strain in the reinforcement (i.e. the ‘local strain’) and the average tension strain in the concrete (i.e. the ‘global strain’). This is illustrated in Figure 5.30, which shows the average global tension strain and local reinforcement tension strain of specimens P07 and P08. The average global tension strains are equal to the tension displacement divided by the height of the specimen and the local tension strains were determined from the strain gauges located at mid-height of each specimen. Test specimens P07 and P08 developed seven and nine cracks respectively. It can be seen in Figure 5.30 that specimen P08, which developed the most cracks, had the least amount of tension-stiffening (i.e. it had the smallest ratio of local to global strain). Whereas specimen P07, which developed the least number of cracks, had the highest amount of tension-stiffening (i.e. it had the highest ratio of local to global strain).

![Figure 5.30: Comparison of local and global strain in test specimens P07 and P08.](image)

Tension stiffening should not be confused with tension shift effect; while similar sounding, each refers to a different concept. Tension shift effect refers to a widely observed behavioural aspect of RC walls, beams and columns where shear stresses in the element cause the flexural cracks, which would otherwise be horizontal, to be inclined (i.e. angled) down towards the compression toe of the element. This results in the “plane-sections remain plane” assumption being somewhat false. These inclined cracks allow the local reinforcement tension strains that develop from the bending moment at a given location in the element to develop at a higher location than the respective bending moment actually occurs. Essentially shifting the moment distribution further up the height of the element. The reader is directed to [P1, P4] for further detailed discussions.

It is hypothesised that the tension stiffening mechanism affects the results of moment-curvature analyses performed using fibre-element analysis methods, such as that discussed in Section 4.3.2. Fibre-element analysis procedures often assume the concrete to have zero tensile capacity and equilibrium of the cross-section is achieved by balancing the stress distribution according to a strain diagram that is established based on the extreme concrete compressive fibre and extreme tensile reinforcing bar (i.e. the dashed line in Figure 5.31). However, as discussed above, the local strain of the reinforcement and the average global strain of the concrete usually differs due to the tension stiffening effect. The curvature of the section should be based on the average global behaviour of overall cross-section. Meaning an over-estimated strain diagram and curvature will be assumed, as indicated in Figure 5.31.
This section will present the development and validation of a tension stiffening model for limited ductile RC walls, which has been validated using the boundary element prism testing presented at the beginning of this chapter. The tension stiffening model developed herein will allow for the local reinforcement tension strains to be converted to global average tension strains, or vice versa. This tension stiffening model could potentially be incorporated into a modified fibre-element analysis procedure, which would allow for the actual curvature of the cross-section to be determined, as opposed to the overestimated curvatures that otherwise result (Figure 5.31). This concept is investigated further in Chapter 8.

![Diagram of tension stiffening in RC walls](image)

**Figure 5.31**: Tension stiffening effect in RC walls.

### 5.6.1 Development of a Tension Stiffening Model for Limited Ductile RC Walls

A simple model for the global average strain based on the local reinforcement strain for limited ductile RC elements has been developed using a generalised crack width model, as presented in Figure 5.32 and Equations 5.3 to 5.7. The model firstly determines an average crack spacing of the element using an assumed average bond stress model and ultimate tensile capacity of the concrete. The crack widths are then calculated by integrating the elastic/inelastic strain diagram, which is determined using the average bond stress model. The strain in the reinforcement is calculated using a non-linear stress-strain reinforcement model. Each aspect of the model will be discussed in the following four sub-sections.

#### 5.6.1.1 Average Crack Spacing

The average crack spacing (i.e. \(a_{ave}\)) is depending on the minimum cracking spacing (i.e. \(a_{min}\)) of the end region of the wall (e.g. the boundary element of the wall). For a rectangular wall the boundary element can be defined in accordance with Eurocode 2 [X10] as the portion of wall extending a minimum of \(0.15L_w\) and \(0.5b_w\) from the end of the wall. Alternatively, for walls with engaged columns or box-shaped building cores, the boundary elements would be the column and flange section of the wall respectively. Each of these three scenarios is illustrated in Chapter 3, Figure 3.15.
Figure 5.32: Generalised crack width and local strain model.

If $\varepsilon_{s,2} \leq \varepsilon_{s,1} \leq \varepsilon_{sy}$:

$$x = 2 \times \left[ \int_0^{L_b} \epsilon(L) \, dL \right]$$

$$= 2 \times \left[ 0.5(\varepsilon_{s,1} + \varepsilon_{s,2})L_b \right] \quad \ldots \text{5.3}$$

If $\varepsilon_{s,1} \leq \varepsilon_{sy} \leq \varepsilon_{s,2}$:

$$x = 2 \times \left[ \int_0^{L_b} \epsilon(L) \, dL + \int_{L_b}^{L_b + L'_b} \epsilon(L) \, dL \right]$$

$$= 2 \times \left[ 0.5(\varepsilon_{s,1} + \varepsilon_{sy})L_b + 0.5(\varepsilon_{sy} + \varepsilon_{s,2})L'_b \right] \quad \ldots \text{5.4}$$

If $\varepsilon_{sy} \leq \varepsilon_{s,2} \leq \varepsilon_{s,1}$:

$$x = 2 \times \left[ \int_0^{L'_b} \epsilon(L) \, dL \right]$$

$$= 2 \times \left[ 0.5(\varepsilon_{s,1} + \varepsilon_{s,2})L'_b \right] \quad \ldots \text{5.5}$$

$$\varepsilon_{local} = \varepsilon_{s,2} \quad \ldots \text{5.6}$$

$$\varepsilon_{global} = \frac{x}{a_{ave}} \quad \ldots \text{5.7}$$

Where:
- $x$ = crack width
- $L_b$ = elastic bond length (i.e. reinf. has not yielded)
- $L'_b$ = inelastic bond length (i.e. reinf. has yielded)
- $u_b$ = average elastic bond strength
- $u'_b$ = average inelastic bond strength
- $a_{ave}$ = average crack spacing
The minimum crack spacing of RC element is the minimum distance required to transfer sufficient force through bond stress from the reinforcement into the concrete such that the tensile capacity of the concrete is exceeded. This can be determined by equating the ultimate tensile capacity of the concrete and the surface area of reinforcement multiplied by the average maximum elastic bond stress (i.e. \( u_b \)), as shown in Equation 5.8.

\[
A_c f'_{ct} = a_{min} u_b d_b \pi \\
a_{min} = \frac{A_c f'_{ct}}{u_b d_b \pi} \quad \ldots \quad 5.8
\]

Where:  
- \( A_c \) = net area of concrete  
- \( f'_{ct} \) = ultimate tensile capacity of concrete  
- \( d_b \) = diameter of reinforcing bar

The net area of concrete is equal to the gross area of concrete (i.e. \( A_g \)) minus the cross-sectional area of vertical reinforcement (i.e. \( A_{st} \)). Further, the gross area of concrete can be expressed in terms of the vertical reinforcement percentage, i.e. \( p_v = A_{st} / A_g \rightarrow A_g = A_{st} / p_v \). If it is assumed that the tensile forces are distributed from the reinforcement to the concrete evenly amongst the vertical bars and that all the vertical bars are the same diameter, the net area of concrete can be expressed in terms of the vertical reinforcement percentage and the bar diameter of the vertical reinforcement, as presented in Equation 5.9.

\[
A_c = \frac{A_{st}}{p_v} - A_{st} = A_{st} \left( \frac{1}{p_v} - 1 \right) = \frac{\pi d_b^2}{4} \left( \frac{1}{p_v} - 1 \right) \quad \ldots \quad 5.9
\]

Using the expression for the net area of concrete presented in Equation 5.9, the minimum crack spacing can be expressed in terms of the ultimate tensile capacity of the concrete, vertical reinforcement bar diameter, average maximum elastic bond stress and percentage of vertical reinforcement (i.e. Equation 5.10).

\[
a_{min} = \frac{f'_{ct} d_b}{4 u_b} \left( \frac{1}{p_v} - 1 \right) \quad \ldots \quad 5.10
\]

Equation 5.10 can be used to calculate the minimum crack spacing in an RC element, however the average crack spacing may in fact be somewhat larger than this value. When an RC element is loaded in tension it initially cracks at discrete irregular locations, with subsequent cracks occurring at a distance of \( a_{min} \) away from these initial cracks. When the spacing of the initial cracks (i.e. \( a \) in Figure 5.33) is greater than \( 2a_{min} \), an additional crack will form as indicated in Figure 5.33. Alternatively, when the spacing of the initial cracks is less than \( 2a_{min} \), an additional crack will not be able to form. As such, Park and Paulay [P2] suggest the "crack spacing of an RC element in tension" can be expected to vary between \( a_{min} \) and \( 2a_{min} \), with an average spacing of approximately \( 1.5a_{min} \).

For this study, an average crack spacing of \( 1.2a_{min} \) has been adopted, resulting in Equation 5.11.

A smaller value of 1.2, as opposed to the average value of 1.5 discussed above, was adopted as it is hypothesised that in the case of RC walls, which have a varying moment diagram with the maximum moment typically occurring at the base of the wall, the first initial crack will occur at the base, corresponding to the maximum moment. Subsequent cracks will then distribute vertically up the wall accordingly. However due to the random nature of concrete, the cracking
would not distribute up the wall at a spacing of exactly $a_{\text{min}}$. A value of 1.2 also corresponds very well to the crack spacing observed in test specimens P02 to P04 and P06 to P17, as presented later in Section 5.6.2 (refer Figures 5.37 and 5.38).

$$a_{\text{ave}} = 1.2a_{\text{min}} = 1.2\frac{f_{\text{ct}}'}{4u_b}\left(\frac{1}{p_\nu} - 1\right)$$ \hspace{1cm} \ldots 5.11

The reader should note that the horizontal reinforcement in RC walls can act as crack propagators and in effect, change the average crack spacing from what is discussed here. Irrespective of this, the crack spacing should still be between $a_{\text{min}}$ and $2a_{\text{min}}$. The spacing of horizontal reinforcement in an RC wall would vary greatly on a case by case basis and including it as a parameter in a generalised model would add significant layers of complexity. This is noted however, as an area for future investigation and research.

![Initial and subsequent cracking in an RC element subjected to axial tension.](image)

**Figure 5.33:** Initial and subsequent cracking in an RC element subjected to axial tension.

### 5.6.1.2 Tensile Strength Concrete

The ultimate tensile capacity of concrete is an important parameter that directly controls the crack spacing. The ultimate tensile capacity of concrete is usually expressed as a function of the concrete’s ultimate compressive capacity. Further, most concrete standards and codes present the ultimate tensile capacity of control in terms of the direct tensile strength and the flexural tensile strength. The latter is for elements where the tensile stresses are due to bending/flexural actions. While the former is for concrete elements that are subjected to uniform tensile stresses across its whole cross-section. The flexural tensile strength is usually higher than the direct tensile strength and is due to the varying stress gradient across the depth of the section that allows the concrete to sustain a somewhat higher level of tensile stress. However, as the depth of the section increases (e.g. the wall length increases), the flexural tensile strength approaches and eventually equals the direct tensile strength, due to the relative rate of change across the stress gradient of the section being reduced. Eurocode 2 [X10] proposes this occurs when the section depth is 1600 mm or greater. It would be unusual for walls to have a length less than about 1500 mm and as such, for a generalised model, it is being proposed that the direct tensile strength be used.

A summary of different equations provided by various codes of practice around the world for calculating the ultimate direct tensile strength of concrete is presented in Table 5.14. This summary includes: The Australia Standard for Concrete Structures, AS 3600 [X6]; The New Zealand Standard and Commentary for Concrete Structures, NZS 3101 Part 1 [X8] and NZS 3101 Part 2 [X38] respectively; Eurocode 2, EN 1992-1-1 [X10]. The recommendations made in NZS 3101 Part 2 are the same as what is proposed in the CEB-FIP Model Code 2010 [X39] and similarly, the recommendations made in EN 1992-1-1 are the same as what is proposed in the fib Model Code for Concrete Structures 2010 [X40].
It should be noted that some codes express the characteristic direct tensile strength (i.e. $f'_{ct}$) in terms of the characteristic compressive strength (i.e. $f'_c$), while others express it in terms of the mean direct tensile strength (i.e. $f_{ctm}$) or mean compressive strength (i.e. $f_{cm}$).

**Table 5.14:** Comparison of direct tensile strength models.

<table>
<thead>
<tr>
<th>Standard</th>
<th>$f'_{ct}$</th>
<th>$f_{ctm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS 3600</td>
<td>$0.36\sqrt{f_c'}$</td>
<td>$1.4f'_{ct} = 0.5\sqrt{f_c'}$</td>
</tr>
<tr>
<td>NZS 3101 Part 1</td>
<td>$0.36\sqrt{f_c'}$</td>
<td>-</td>
</tr>
<tr>
<td>NZS 3101 Part 2 (CEB-FIP model code 1990)</td>
<td>$0.68f_{ctm}$</td>
<td>$1.4\left(\frac{f'_c}{10}\right)^{2/3}$</td>
</tr>
<tr>
<td>EN 1992-1-1 (fib model code 2010)</td>
<td>$0.7f_{ctm}$</td>
<td>$f'<em>{c} \leq 50$ MPa: $0.3(f'<em>c)^{2/3}$ $f'</em>{c} &gt; 50$ MPa: $2.12\ln \left(1 + \frac{f</em>{cm}}{10}\right)$</td>
</tr>
</tbody>
</table>

As discussed in Section 4.4.3, Foster et al. [F1] proposed that the average mean compressive strength to characteristic compressive strength ratio (i.e. $f_{cm}/f'_c$) for concrete grade N40 in Australia is 1.215. This was based on approximately 4,600 concrete cylinder test reports undertaken in 12 locations across of Australia. Using the AS 3600 proposed equations, the mean direct tensile strength could then be expressed by Equation 5.12 for grade N40 concrete.

$$f_{ctm} = 0.46\sqrt{f_{cm}} \quad \cdots 5.12$$

Figure 5.34 presents the ratio of the maximum tensile strength (i.e. $f_{ctm}$) of test specimens P02, P03 and P07 to P17 divided by the square root of the compressive strength of the respective specimen (i.e. $f_{cm}$). If the type 1 round two test specimens (i.e. P11 to P17) are neglected, due to some of the compressive strengths being much lower than anticipated (refer discussion in Section 5.1.2.2), the average ratio from Figure 5.34 is 0.40. This is slightly lower than what is being proposed in Equation 5.12 above.

**Figure 5.34:** Test specimens P02, P03 and P07 to P17 tensile strength.
5.6.1.3 Bond Stress Between the Concrete and Reinforcement

A simplified bond stress model was adopted where it is assumed that the bond stress is constant along the length of the bar bonded to the concrete. The model presented in the fib Bulletin 43, *Structural Connections for Precast Concrete Buildings* [X41] was adopted. The fib Bulletin 43 proposed Equation 5.13 for the average elastic bond stress, where it is taken as the maximum bond stress from the widely used CEB-FIP Model Code 1990 [X39] bond-slip model (i.e. Figure 5.35, Equation 5.14 and Table 5.16) and multiples it by the factor $\alpha_1$, which is dependent on the bar diameter of the reinforcement (refer Table 5.15).

$$u_{b,\text{ave}} = \alpha_1 u_{b,\text{max}}$$

Where: $\alpha_1 =$ refer Table 5.15

*Table 5.15:* Factor to convert maximum bond stress to average bond stress [X41].

<table>
<thead>
<tr>
<th>$d_b$</th>
<th>6</th>
<th>8</th>
<th>10</th>
<th>12</th>
<th>16</th>
<th>20</th>
<th>25</th>
<th>32</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_1$</td>
<td>0.30</td>
<td>0.32</td>
<td>0.34</td>
<td>0.36</td>
<td>0.40</td>
<td>0.42</td>
<td>0.45</td>
<td>0.45</td>
</tr>
</tbody>
</table>

$$u_b = u_{b,\text{max}} \left( \frac{s_b}{s_{b,1}} \right)^{\alpha_2}$$

*Table 5.16:* CEB-FIP Model Code 1990 bond-slip model [X39].

| $s_{b,1}$ | 0.6 mm | 0.6 mm | 1.0 mm | 1.0 mm |
| $s_{b,2}$ | 0.6 mm | 0.6 mm | 3.0 mm | 3.0 mm |
| $s_{b,3}$ | 1.0 mm | 2.5 mm | clear rib spacing | clear rib spacing |
| $\alpha_2$ | 0.4 | 0.4 | 0.4 | 0.4 |
| $u_{b,\text{max}}$ | $2.0 \sqrt{f'_c}$ | $1.0 \sqrt{f'_c}$ | $2.5 \sqrt{f'_c}$ | $1.25 \sqrt{f'_c}$ |
| $u_{b,f}$ | $0.15 u_{b,\text{max}}$ | $0.15 u_{b,\text{max}}$ | $0.40 u_{b,\text{max}}$ | $0.40 u_{b,\text{max}}$ |

* Bond failure by splitting of the concrete.
† Bond failure by shearing of the concrete between the ribs.

The unconfined concrete bond stress parameters were adopted from Table 5.16, given the model was being developed for limited ductile RC walls, which are generally detailed with little or no cross-ties or confinement. The fib Bulletin recommends that the ‘good bond conditions’ in Table 5.16 be adopted if while casting, the bar is on a 45° to 90° angle to the horizontal or the bar is at angle below 45° and it does not have more than 250 mm of concrete poured below the bar. Otherwise, the ‘all other bond conditions’ in Table 5.16 should be adopted.
The bond-slip model and average elastic bond stress proposed is only valid up until yielding of the reinforcement occurs. The amount of bond stress between the concrete and the reinforcement will decrease after yielding of the rebar occurs. The fib Bulletin 43, with a citation to a study by Engström [E5], proposes that the average bond stress in the regions where yielding of the reinforcement has occurred can be taken as 0.27 times the maximum bond stress (i.e. Equation 5.15). However, for the conditions being assumed (i.e. unconfined concrete), Equation 5.15 will result in an average inelastic bond stress value greater than the $u_{bf}$ bond stress value in Table 5.16, which occurs after only mere 1.0 mm of slip has occurred. Therefore, it was deemed that Equation 5.15 was inappropriate and the $u_{bf}$ bond stress value in Table 5.16 was adopted as the inelastic bond stress in the proposed model, i.e. $u'_b = u_{bf}$.

This results in the inelastic bond stress ranging from 0.42$u_b$ for 12 mm diameter bars down to 0.36$u_b$ for 20 mm diameter bars and 0.33$u_b$ for 32 mm diameter bars. Other studies (i.e. [F3, S10]) have reported the average inelastic bond stress as simply 50% of the average elastic bond stress, i.e. $0.5u_b$. However, it is believed 50% is too generous and the slightly lower values proposed provide better correlation with test results, as presented in Section 5.6.2.

$$u'_b = 0.27 \times u_{b,max} \quad \ldots \ 5.15$$

### 5.6.1.4 Model Assumptions

An underlying assumption of the model developed is that cracking occurs prior to the development of inelastic deformation of the vertical reinforcement. That is, the tensile capacity of the concrete in the boundary element (i.e. $A_{c, f_{ctm}}$) has to be less than the tensile yield capacity of the vertical reinforcement within the boundary element (i.e. $A_{v, f_{sy}}$). Therefore, Equation 5.16 must be satisfied, if the ensuing model in Section 5.6.3 is used.

$$p_{v, min} = \frac{f_{ctm}}{f_{sy}} \quad \ldots \ 5.16$$

In addition to ensuring the assumptions of this model are met, providing enough vertical reinforcement such that the condition of Equation 5.16 is met, will ensure the RC wall being designed will be capable of developing distributing cracking, which is essential for the development of rational and desirable yielding mechanism in the wall. This is discussed in greater detail in Section 3.5.4.
5.6.2 Tension Stiffening Model Validation

The tension stiffening modelling process for limited ductile RC walls was validated against the boundary element test specimens (i.e. P02 to P04 and P06 to P17). Initially the proposed crack spacing and average elastic bond stress equations (i.e. Equations 5.11 and 5.13) were compared against the crack distributions observed in the test specimen, which are presented in Figures 5.37 and 5.38. It should be noted that the crack distributions shown in Figures 5.37 and 5.38 are the 'average' crack distributions of each specimen. The cracks were often not perfectly horizontal or split into two separate cracks near one or either end of the specimen. Therefore, the crack distributions were approximated with an equivalent set of horizontal cracks at the spacings shown in Figures 5.37 and 5.38.

The minimum crack spacing for each test specimen was calculated using Equation 5.10, the tensile strength of the respective specimen (given in either Table 5.3, 5.6 or 5.9) and the average elastic bond stress using Equation 5.13. These values were compared against the average crack spacing for each test specimen, as shown in Figure 5.36. The error bars in Figure 5.36 represent the maximum and minimum observed crack spacing in each respective specimen. It shown here that good correlation was observed between the calculated and average crack spacing for each test specimen. Further, the average ratio of observed average crack spacing and calculated (i.e. theoretical) minimum crack spacing of the specimens was equal to 1.1. This therefore provides good validation of the crack spacing and elastic average bond stress models proposed above in Section 5.6.1.

![Figure 5.36: Comparison between calculated and average crack spacing.](image)

The behaviour of test specimen was then modelled to compare the proposed model against the respective specimen's actual performance. The modelling procedure was a force-based procedure where an initial force was assumed for a specimen, which was converted to an average stress in the vertical reinforcement. The individual crack width for each crack shown in Figures 5.37 to 5.38 were calculated using Equations 5.3 to 5.5. The overall displacement of the specimen, for that particular load step, was calculated by summing all the crack widths. The initial force is then increased and the process is repeated to determine the next displacement point. This procedure is repeated until all the desired displacements have been calculated. The stress distribution in the reinforcement is calculated using the average elastic and inelastic bond stress models discussed previously and then strain distribution in the reinforcement is calculated using a non-linear stress-strain relationship (which are presented in Appendix B for each specimen).
This procedure is illustrated in Figures 5.39 and 5.40 for test specimens P08 and P10 respectively. Figures 5.39(b) and 5.40(b) respectively, show both the strain and stress distributions along the vertical reinforcement for the displacement increment equivalent to 1% average global strain in the specimen.

The force-displacement response for test specimens P08 and P10 are presented in Figures 5.39(d) and 5.40(d) respectively. The reaming force-displacement response curves for test specimens P02 to P04, P07, P09 and P011 to P17 are presented in Figure 5.41. Very good correlation between the response of each specimen and the respective theoretical behaviour predicted by the model can be seen in these figures. Of particular note is specimens P08 and P10, where both the force-displacement behaviour (i.e. Figures correlates 5.39(d) and 5.40(d) respectively) and the measured strain local strain using strain gauges (i.e. Figures correlates 5.39(c) and 5.40(c) respectively) matches very well with the predicted behaviour. This therefore provides good validation of the proposed modelling approach.

It should be noted that the predicted behaviour for some of the specimens with smaller diameter reinforcing bars, i.e. specimens P07, P09 and P13 that have N10 vertical bars (refer Figures 5.41(d), 5.41(e) and 5.41(h) respectively), do not have as good a correlation as the specimens with the larger diameter reinforcing bars, i.e. specimens P08 and P10 that have N16 vertical bars. This is due to the inelastic bond stress relationship assumed for the 10 mm bars. However, given that N10 bars are rarely used in RC walls, the assumed inelastic bond stress model is still appropriate.

![Figure 5.37: Average crack distribution observed in the type 1 test specimens.](image-url)
Figure 5.38: Average crack distribution observed in the type 2 test specimens.
Figure 5.39: Tension stiffening model validation – test specimen P08.

Figure 5.40: Tension stiffening model validation – test specimen P10.
Figure 5.41: Tension stiffening model validation – all other test specimens.
5.6.3 Parametric Study and A Proposed Global to Local Strain Relationship

A parametric study was performed using the tension stiffening model developed above to determine a relationship between global average strain and local reinforcement strain for D500N reinforcement, which is the standard grade of rebar in Australia for detailing RC walls. The proposed relationships for N12, N16, N20, N24, N28 and N32 reinforcement are presented in Figures 5.44(a) to 5.44(f).

The mean response bilinear stress-strain curve for D500N proposed in Section 4.4.3 was adopted for the analysis, as shown in Figure 5.42. The mean response relationship (as opposed to the characteristic ‘code’ material properties) was adopted since a global strain to local strain relationship is only of interest when the actual in-situ behaviour of an RC element is being considered. Interestingly enough, the relationship is essentially insensitive to concrete strength, as highlighted in Figure 5.43. The main factors affecting the relationship is the percentage of vertical reinforcement (i.e. $p_v$) and the mean ultimate tensile strength of the element (i.e. $f_{ctm}$). The bar diameter effects the relationship also, but to a much lesser extent. The proposed relationships in Figure 5.44 are presented in terms of the constant $k_1$, which is equal to the percentage of vertical reinforcement divided by the mean ultimate tensile strength of the element, i.e. $k_1 = p_v / f_{ctm}$.

![Figure 5.42: Mean response stress-strain curve for D500N reinforcement.](image)

![Figure 5.43: Concrete strength effect of global to local strain relationship.](image)
Figure 5.44: Global average strain versus local reinforcement strain.

(a) N12 reinforcement
(b) N16 reinforcement
(c) N20 reinforcement
(d) N24 reinforcement
(e) N28 reinforcement
(f) N32 reinforcement
5.7 Conclusions

This chapter has presented the details and results of the first experimental testing program. Seventeen boundary element prism specimens were constructed and tested under cyclic axial tension-compression loading, which is meant to simulate the equivalent type of loading the end regions of RC walls are subjected to during system level response. The testing program included both high and low slenderness ratio (i.e. $H_w/t_w$) specimens. The specimens captured many different failure mechanisms of RC walls, particularly the global out-of-plane buckling and local bar buckling of vertical reinforcement failure modes, which were both widely observed in recent devastating earthquakes that occurred in Christchurch (2011) and Chile (2010).

It was observed that RC elements under cyclic axial tension-compression loading undergo a bifurcation effect, which is dependent on the amount of inelastic plastic strain the vertical reinforcement is subjected to during the previous reversed cyclic tension displacement cycle. After load reversal and the element is subjected to axial compression, where the element will either ‘overcome’ the residual plastic tension strains and regain its initial axial stiffness or the residual plastic tension strains will initiate a global out-of-plane buckling or local bar buckling failure mechanism, which results in axial load failure of the element. The axial load failure occurs at a load equal to about 10% of the gross compression capacity of the element.

The results of the testing suggest that RC walls can sustain local tension strains in the reinforcement of about 6–7% before bar buckling occurs in the subsequent reversed load cycle. While out-of-plane buckling modes were observed, further research is still required before a comprehensive model can be proposed to predict the tension strains required to initiate the failure mode in the subsequent reversed load cycle. It was also shown that strain rate seems to have little effect on the out-of-plane buckling mechanism, meaning that the pseudo-static loading protocols used for the test specimens were appropriate for experimentally assessing this behaviour. Finally, singularly reinforced specimens behaved poorly in comparison to the specimens detailed with two layers of vertical bars (i.e. one per face of the wall). The singularly reinforced specimens were subject to local bar buckling after lower local tension strain values in previous reversed load cycles and also had larger out-of-plane movement relative to the ‘doubly’ reinforced specimens.

A series of cyclic tension-compression rebar tests were also performed. These tests showed that cyclic loading does not affect (i.e. reduce) the ultimate stress or strain of the bar, unless the bar buckle in the previous compression cycle. Where minor buckling occurred in the compression cycles, the ultimate strain (i.e. uniform elongation) was reduced to about 60% of the monotonic value.

The results of the boundary element prism testing were used to develop and propose a set of reinforcement and concrete strain limits, which could be used in displacement-based assessment procedures of limited ductile RC walls.

The chapter is concluded with the development of a tension stiffening model for limited ductile RC walls. The model was validated against the boundary element prism specimens and very good correlation was observed. The model allowed for the development of a global average strain to local reinforcement strain relationship through the use of a parametric study.
Chapter 6

Experimental Testing of Cast In-Situ Limited Ductile RC Walls

This chapter presents an overview, results and discussion on the second program of large-scale experimental testing performed on RC walls. The second program of experimental work consisted of two monolithic cast in-situ wall tests. The first test specimen was a slender rectangular wall and the second was a box shaped building core specimen. The test specimens were designed and detailed to best match standard industry practices (as documented in Chapter 3), which broadly speaking, was shown to be high aspect ratio (i.e. slender) walls with limited ductile reinforcement detailing. The testing was performed using the MAST System, a state-of-the-art large-scale structural testing machine in the Smart Structures Laboratory at Swinburne University of Technology.

6.1 Experimental Testing Program

The first test specimen, denoted S01, was a rectangular RC wall and the second test specimen, denoted S02, was a box-shaped building core specimen (Figure 6.1), which were the two most common wall sections identified in the reconnaissance survey documented in Chapter 3. The properties of each specimen (e.g. reinforcement ratio, shear span ratio, axial load ratio etc.) were selected to best match typical design and detailing practices used in industry, also identified in Chapter 3. The specimens were initially designed to represent a 60% to 70% full-scale ground storey wall in a real building, however the ideal geometry was constrained by the test machine.

Both specimens were generally detailed in accordance with the main body of AS 3600 [X6], which typical results in a limited ductile classification to AS 1170.4 [X4]. Each specimen had a constant spaced gird of vertical and horizontal reinforcement on each face of the wall with lapped horizontal 'U' bars at the end regions of the rectangular specimen and corner interactions of the building core specimen (Figure 6.1). The specimens were detailed to have a moderate percentage of vertical reinforcement, as summarised in Table 6.1.
Figure 6.1: Test specimen S01 and S02 details.
Chapter 6: Experimental Testing of Cast In-Situ Limited Ductile RC Walls

Table 6.1: Test specimen properties (cast in-situ wall tests).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Shear-span ratio*</th>
<th>Vertical reinf. ratio</th>
<th>Horizontal reinf. ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>6.5</td>
<td>0.018</td>
<td>0.005</td>
</tr>
<tr>
<td>S02</td>
<td>6.5</td>
<td>0.014</td>
<td>0.005</td>
</tr>
</tbody>
</table>

* Shear span-ratio equals \( \frac{M^*}{V^*L_w} \).

The walls were constructed using D500N reinforcing bars to AS/NZS 4671 [X29], which have a minimum characteristic yield stress, strain hardening ratio and ultimate strain of 500 MPa, 1.08 and 5% respectively. The actual in-situ material properties of the reinforcement used for each test specimen is summarised in Table 6.2. For each entry in Table 6.2, a minimum of four tensile tests of rebar samples was performed.

Table 6.2: Reinforcement properties (cast in-situ wall tests).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement</th>
<th>( f_{sy} )</th>
<th>( f_{su} )</th>
<th>( \frac{f_{sy}}{f_{su}} )</th>
<th>( \varepsilon_{su} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>N20 vertical bars</td>
<td>532 MPa</td>
<td>637 MPa</td>
<td>1.20</td>
<td>12.6%</td>
</tr>
<tr>
<td>S01</td>
<td>N12 horizontal bars</td>
<td>553 MPa</td>
<td>706 MPa</td>
<td>1.28</td>
<td>12.7%</td>
</tr>
<tr>
<td>S02</td>
<td>N12 vertical bars</td>
<td>544 MPa</td>
<td>698 MPa</td>
<td>1.28</td>
<td>11.0%</td>
</tr>
<tr>
<td>S02</td>
<td>N10 horizontal bars</td>
<td>545 MPa</td>
<td>680 MPa</td>
<td>1.25</td>
<td>12.1%</td>
</tr>
</tbody>
</table>

Note: N10, N12 and N20 denotes 10 mm, 12 mm and 20 mm nominal diameter grade D500N reinforcing bars to AS/NZS 4671 respectively.

The specimens were constructed using standard N40 grade concrete, which has a minimum characteristic 28-day compressive cylinder strength of 40 MPa. The maximum aggregate size of the concrete mix was 14 mm. The concrete strengths for test specimens S01 and S02 are summarised in Table 6.3. The concrete strength for S01 was determined from cylinder compression tests performed by the supplier, which were performed on 100 mm diameter and 200 mm long specimens that were cured continuously in a water bath. These curing conditions of the cylinders would result in the in-situ concrete strength of the specimen being marginally lower than the value presented in Table 6.3. For analysis purposes, the in-situ concrete strength of the specimen could approximately be taken as 90% of the mean cylinder strength (as discussed previously in Section 4.4.3.2), which would results a maximum compressive strength of 37.7 MPa. The concrete strength for S02 was determined from core samples taken from the upper section of the specimen in locations where no cracking was presented and linear elastic behaviour of the concrete was observed. Therefore the value presented in Table 6.3 is considered to be representative of the actual in-situ strength of the specimen. However, it should be noted that the strength of test specimen S02 on test day was significantly lower than the specified strength.

Table 6.3: Concrete strength (cast in-situ wall tests).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specified strength</th>
<th>Actual strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>40 MPa</td>
<td>41.9 MPa</td>
</tr>
<tr>
<td>S02</td>
<td>40 MPa</td>
<td>31.6 MPa</td>
</tr>
</tbody>
</table>
AS 3600 does not provide any restrictions on the method or location of splicing vertical reinforcement in walls and has resulted in a standard industry practice where the majority of walls are detailed and constructed with lap splices of the vertical reinforcement at the base of the wall, typically in the plastic hinge zone, generally for ease of construction (refer Chapter 3). As such, the test specimens were detailed and constructed with a lap splice of the vertical reinforcement at the base of the wall in the plastic zone. The lap splice length was calculated in accordance with AS 3600. The lap splice for specimen S01, which was detailed using N20 (i.e. 20 mm nominal diameter grade D500N rebar), was 900 mm and for specimen S02, which was detailing using N12 (i.e. 12 mm nominal diameter grade D500N rebar), was 500 mm (Figure 6.2). The construction drawings for each test specimen are provided in Appendix A.

![Figure 6.2: Test specimen S01 and S02 cross section (Y-Y sections).](image)

The test specimens were designed to represent the ground floor component of a four-storey wall and building core respectively, as shown in Figure 6.3. The bending moment and shear force response of the ground floor component of a taller four storey wall is being simulated on the one storey test specimen using an applied lateral force and moment at the top of the specimen. To simulate this equivalent response the moment is applied as a function of the lateral force multiplied by a constant \( k \). The constant \( k \) is dependent on (i) the number of stories in the building, (ii) the inter-storey height of the building and (iii) the profile of the lateral load. For a four-storey element with an inter-storey height of 2600 mm and a triangular lateral load distribution the constant \( k \) equals 5.2. This results in the test specimens having a shear-span ratio of 6.5. The shear-span ratio is equal to the moment at the base of the wall divided by the product of the shear force and wall length, i.e. \( M^*/(V^*L_w) \). Alternatively put, the shear-span ratio is equal to the aspect ratio of the equivalent single degree of freedom system, \( H_e/L_w \). The formula for calculating the equivalent force and moment on the one storey test wall for a triangular lateral load profile are presented in Equations 6.1 to 6.3.
**Figure 6.3:** Simulation of a four-storey building response using the one-storey test specimen.

\[
F' = \sum_{i=1}^{n} F_i \\
M' = kF' \\
k = \frac{h_S \sum_{i=2}^{n} i(i-1)}{\sum_{i=1}^{n} i} 
\]

Where:  
- \( h_S \) = floor-to-floor height  
- \( i \) = \( i \)-th storey  
- \( n \) = number of stories

### 6.2 Instrumentation

A combination of physical instrumentation attached to the test specimens – consisting of linear variable displacement transducers (LVDTs), string potentiometers and laser displacement sensors – and a contactless photogrammetry system were used to monitor and measure the behaviour and response of the test specimens. The physical instrumentation was primarily used to measure the overall global behaviour (e.g. in-plane displacements, rotations and axial behaviour) and the photogrammetry system was primarily used for quantifying the different types of deformations (e.g. flexure, shear and sliding) and sectional responses (e.g. strain and curvature distributions) of the specimens.
The photogrammetry system used was the V-STARS N series by Geodetic Systems. The heavy reliance on photogrammetry measurement systems instead of traditional physical instrumentation, as traditionally adopted in RC wall testing, is reflective of a shift in philosophical mindset of the laboratory staff and researchers in the structures department at Swinburne University of Technology, which meant resources for additional physical instrumentation, above what used, was not readily available. The V-STARS system is a turnkey single camera photogrammetry system, which can be used to make discrete measurements of the test specimen while the testing procedure is paused. This is in contrast to the physical instrumentation which is recording data continuously for the whole duration of testing. The advantage of the former is that much more strain and displacement data for the specimen can be determined at that point in time compared to the latter. The system requires the user to take a series of photos of the targeted object (i.e. test specimen) from multiple orientations and points of view, which are then post processed using the V-STARS computer software to create a digital model of the targeted object’s geometry, from which the x-y-z movement of each target in 3-dimensional space is ascertained.

The location of photogrammetry targets, where the x-y-z movement was calculated and recorded at discrete moments throughout the test, for each specimen, is shown in Figure 6.5. It should be noted that the set-out dimensions of the targets in Figure 6.5 is approximate only and the exact location of each target is determined using the calibrated photogrammetry system. Verification exercises were performed to ensure the accuracy of the photogrammetry system and are documented and included as part of Appendix C for completeness.

A series of string potentiometers (SPOTs) were used to measure the in-plane lateral displacement, rotation and axial behaviour of the test specimens (Figure 6.6). A series of laser displacement sensors were used to monitor and measure any slip or uplift (i.e. base rotation) of the specimens during the testing (Figure 6.6). Lastly, a series of LVDTs, stacked vertically at each end of the wall, were used to verify the strain and curvature distributions determined from the photogrammetry system (Figures 6.6 and 6.4). A summary table of all the physical instrumentation, including transducer details such as stroke length, is provided in Appendix C.

![Figure 6.4: Right – LVDT stack specimen S01. Left – LVDT stack specimen S02.](image)
Figure 6.5: Test specimen S01 and S02 photogrammetry targets (approximate locations).

specimen S01 elevation A
specimen S02 elevation A
specimen S02 elevation B
specimen S02 elevation C
specimen S01 section A-A
specimen S01 section B-B
specimen S02 section C-C
specimen S02 section D-D
6.3 Test Setup and Loading Protocol

The test specimens were tested using the MAST System under a cyclic quasi-static unidirectional loading regime, as described in the following two sub-sections, i.e. Sections 6.3.1 and 6.3.2.

6.3.1 The MAST System

The specimens were tested using the Multi-Axis Substructure Testing (MAST) System in the Smart Structures Laboratory (SSL) at Swinburne University of Technology. The MAST System is a state-of-the-art test machine capable of applying full six degree-of-freedom (DOF) loading in mixed-mode, switched-mode, hybrid or a combination therein [H11]. The MAST controller uses MTS control hardware, MTS 793 Degree of Freedom software and MTS TestSuite to control the six DOFs using eight individual MTS actuators (i.e. four ±1,000 kN vertical actuators and two pairs of ±500 kN horizontal actuators in each orthogonal directions). The MAST System can test specimens of any material or shape with a maximum plan section of 3x3m, height of 3.35m and weight of 10 tonnes. The MAST System and its non-concurrent DOF force and displacement capacities are summarised in Figure 6.7 and Table 6.4 respectively. The actuator set out is shown in Figure 6.8.
Table 6.4: The MAST System’s non-concurrent individual DOF capacities.

<table>
<thead>
<tr>
<th>Degree of freedom</th>
<th>Force capacity</th>
<th>Displacement limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_x$ – x-axis translation</td>
<td>±1,000 kN</td>
<td>±250 mm</td>
</tr>
<tr>
<td>$T_y$ – y-axis translation</td>
<td>±1,000 kN</td>
<td>±250 mm</td>
</tr>
<tr>
<td>$T_z$ – z-axis translation</td>
<td>±4,000 kN</td>
<td>±250 mm</td>
</tr>
<tr>
<td>$R_x$ – x-axis translation</td>
<td>±4,500 kNm</td>
<td>±6.3°</td>
</tr>
<tr>
<td>$R_y$ – y-axis translation</td>
<td>±4,500 kNm</td>
<td>±6.3°</td>
</tr>
<tr>
<td>$R_z$ – z-axis translation</td>
<td>±3,500 kNm</td>
<td>±8.1°</td>
</tr>
</tbody>
</table>

Figure 6.7: The MAST System at Swinburne University of Technology.

Figure 6.8: Actuator set out drawings of the MAST System.
Three-dimensional computer-generated visualisations of the MAST System are presented in Figures 6.9(a) to 6.9(f), which show the physical non-concurrent individual DOF displacements of the system. Each non-concurrent DOF displacement is shown to scale and at the maximum respective displacement capacity for that DOF.

(a) x-axis translation of +250mm

(b) x-axis rotation of +6.3°

(c) y-axis translation of +250mm

(d) y-axis rotation of +6.3°

(e) y-axis translation of +250mm

(f) z-axis rotation of +8.1°

Figure 6.9: The MAST System's non-concurrent individual DOF displacements.
The specimens are being tested under unidirectional lateral load, requiring a two-dimensional test setup. Under this loading scenario the MAST System’s third dimension actuators would be performing a secondary function of stabilising the two-dimensional test setup (i.e. providing boundary element support conditions). To maximise the capacity of the MAST System the specimens are being tested at a 45-degree angle to the system default axes shown in Figure 6.7. The MTS 793 Degree of Freedom software allows the user to readily move and/or rotate the default axis of the system as required. This two-dimensional test setup with the z-axis rotation of +45° (as shown in Figure 6.10) increases the capacity of the MAST System by roughly a factor of root 2, resulting in a non-concurrent horizontal capacity of ±1,414 kN with a lateral displacement limit of ±354 mm and a non-concurrent moment capacity of ±6,364 kNm with a rotation limit of ±8.9° (as shown in Figure 6.11).

![Specimen S01](image1)

![Specimen S02](image2)

**Figure 6.10:** Test specimen in the MAST System with the 45-degree z-axis rotation.

(a) x-axis translation of +354mm  
(b) y-axis rotation of +8.9°

**Figure 6.11:** The MAST System’s non-concurrent individual DOF displacements with the 45-degree z-axis rotation adopted for the testing regime.
### 6.3.2 Loading Protocol

The specimens were tested under unidirectional quasi-static cyclic test conditions. Initially an axial load was applied to the test specimens to simulate the pre-compression load on the wall (i.e. the gravity load from the surrounding building). The axial load was applied in force-controlled mode in the z-axis ($T_z$) and maintained for the duration of the test until axial load failure of the specimen occurred (i.e. complete structural collapse). The applied axial force for specimen S01 and S02 was -585 kN and -1200 kN respectively. Resulting in the axial load ratio (i.e. axial load divided by the product of the gross cross-sectional area of wall and the compressive strength of the concrete) for specimens S01 and S02 being 6.5% and 7.7% respectively.

After the axial load was applied to the specimen, the specimen was subject to incrementally increasing cyclic lateral displacements in the x-axis ($T_x$). For each lateral displacement increment the specimens were subjected to two positive and two negative cycles, in line with the recommendations given in ACI 374.2R-13 [X42]. The initial displacement increment selected for each specimen was calculated based on the theoretical cracking moment capacity of the cross-section. After the initial set of lateral displacement cycles, the subsequent sets of lateral displacement increments were determined so the next value was between $5/4$ and $3/2$ times the current displacement increment. This procedure for calculating new lateral displacement increments was determined with reference to ACI ITG-5.1-07 [X43]. The test was paused at the second positive and second negative cycle of each increment to take photos, mark cracks and take photogrammetry measurements. The lateral x-axis displacement loading protocols for specimens S01 and S02 are shown in Figure 6.12. The numbers above and below the x-axis displacement values in Figure 6.12 are load cycle numbers and are used as reference points for discussing the results and behaviour of each test specimen in the subsequent sections.

For the duration of the test a moment was applied about the y-axis in force-controlled behaviour to simulate the bending moment and shear force response of a taller four storey wall, with a shear-span ratio of 6.5, in the one storey test specimen (Figure 6.3). The applied moment was equal to the in-plane x-axis force multiplied by a value of 5.2, as discussed in the previous section. The remaining out-of-plane DOFs were commanded to zero displacement and zero rotation in displacement-controlled behaviour for the duration of the test, i.e. $T_y$ was equal to zero movement and $R_x$ and $R_z$ was equal to zero rotation. A summary of the six DOF loading protocol is presented in Table 6.5.

The solid line in Figures 6.12(a) and 6.12(b) denotes the actual x-axis displacements of specimen S01 and S02 respectively, calculated using independently mounted string potentiometers. It can be seen here that the commanded x-axis displacement values, denoted by the horizontal dashed lines in Figure 6.12, were not achieved. The difference between the commanded displacement and actual response of the specimen is an accumulation of (i) sliding at the interface between the bottom of the specimen and the strong floor, (ii) sliding at the interface between the top of the specimen and the underside of the crosshead of the MAST system and (iii) elongation of the bolts at the top and bottom connection points of each of the eight actuators in the system. The majority of the movement was attributed to the latter item. Due to the inherent complexities of the MAST System (e.g. the system consists of eight individual actuators with multiple connection points, steel brackets and bolts), these discrepancies are near impossible to completely eliminate. However, it is noted that in future tests much of this discrepancy could be avoided by commanding the x-axis displacement values using independently mounted instrumentation. Verification that the loading protocol was maintained for the duration of the test is presented in Appendix C.
Table 6.5: The MAST System loading protocol summary – test specimen S01 and S02.

<table>
<thead>
<tr>
<th>Degree of freedom</th>
<th>Mode</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_x$ – x-axis translation</td>
<td>Displacement</td>
<td>Figure 6.12</td>
</tr>
<tr>
<td>$T_y$ – y-axis translation</td>
<td>Displacement</td>
<td>Zero movement</td>
</tr>
<tr>
<td>$T_z$ – z-axis translation</td>
<td>Force</td>
<td>Constant force *</td>
</tr>
<tr>
<td>$R_x$ – x-axis rotation</td>
<td>Displacement</td>
<td>Zero rotation</td>
</tr>
<tr>
<td>$R_y$ – y-axis rotation</td>
<td>Force</td>
<td>$M_y = kF_x$ †</td>
</tr>
<tr>
<td>$R_z$ – z-axis rotation</td>
<td>Displacement</td>
<td>Zero rotation</td>
</tr>
</tbody>
</table>

* The axial force for test specimen S01 and S02 was -585 kN and -1200 kN respectively.
† The specimens were tested with a shear-span ratio of 6.5, which resulted in a $k$ factor of 5.2 (as discussed in Section 6.1 and illustrated in Figure 6.3).

(a) test specimen S01 – rectangular wall

(b) test specimen S02 – building core

Figure 6.12: Cyclic x direction displacement increments – test specimen S01 and S02.
6.4 Test Results

6.4.1 General

Test specimen S01 and S02 both achieved good in-plane lateral response given the basic level of reinforcement detailed provided (e.g. no confinement reinforcement and lap splices of the vertical reinforcement at the base of the wall). The force-displacement behaviour, at the top of the specimen, for test specimens S01 and S02 are presented in Figures 6.13 and 6.16 respectively. The moment-rotation and lateral displacement-axial displacement behaviour, at the top of the specimen, for test specimens S01 and S02 are presented in Figures 6.14 and 6.17 respectively.

The rectangular wall specimen (i.e. S01) and the building core specimen (i.e. S02) were able to undergo +2.1%/-2.5% and +1.5%/-1.5% lateral drift prior to lateral load failure occurring (i.e. the lateral strength of the specimen dropped below 80% of its respective maximum capacity). Following this loss of lateral load capacity, S01 continued to degrade down to zero strength after the second reversed cycle of ~4.2% lateral drift. However, despite this, the wall continued to withstand the initial vertical load of -585 kN while at zero lateral strength. Following this the wall was to be subjected to a larger series of lateral drift values but axial load failure occurred (i.e. complete structural collapse) following a lateral drift of -4.4%.

The rectangular wall specimen (i.e. S01) failed in flexure via crushing of the concrete in the extreme compressive fibre of the section, at the base of the wall. This allowed for the gradual reduction in lateral strength of the wall seen in Figure 6.13. The wall experienced some minor bar buckling of the vertical reinforcement at the base of the wall (Figure 6.15), however this occurred after compression failure of the concrete had begun and the lateral strength of the wall had started to decline. It is believed the bar buckling did not significantly affect the overall behaviour.

![Figure 6.13: Specimen S01 force-displacement behaviour at the top of the specimen.](image-url)
Figure 6.14: Left – specimen S01 moment-rotation behaviour at the top of the specimen. Right – axial displacement-lateral displacement behaviour at the top of the specimen.

Figure 6.15: Bar buckling in test specimen S01 (cycle 115, +2.8% lateral drift).

The building core specimen (i.e. S02) was able to achieve a considerably larger level of in-plane lateral drift after lateral load failure had occurred, prior to axial load failure of the specimen. When the specimen was subjected to displacement increments equal to ±3.3% lateral drift, on the first positive cycle the lateral load capacity dropped to about 55% of the maximum and then on the first reversed negative cycle the lateral load capacity dropped to below 20% of the maximum. Corresponding with this drop in lateral strength, the specimen underwent significant axial shortening, where it went from +15 mm at +3.3% lateral drift to -11 mm at -3.3% lateral drift (Figure 6.17). Despite the serious reduction in lateral strength and axial shortening, the section was still able to resist the initial axial load of -1200 kN. The specimen was then subjected to a larger in-plane lateral drift of +4.8%, followed by -4.6%. At -4.6% drift lateral drift, the specimen was still able to withstand the initial axial load of -1200 kN, however when the specimen was unloaded from -4.6%, axial load failure of the specimen (i.e. complete structural collapse) occurred around -2.9% lateral drift.
Figure 6.16: Specimen S02 force-displacement behaviour at the top of the specimen.

Figure 6.17: Left – specimen S02 moment-rotation behaviour at the top of the specimen. Right – specimen S02 axial displacement-lateral displacement behaviour at the top of the specimen.

S02 failed in flexure via crushing of the concrete in the extreme compressive fibre of the section, at the base of the wall. However, unlike specimen S01, where compression failure was due to the ultimate compression strain of the concrete being exceeded, the compression failure here was due to degradation of the concrete due to a combination of tensile fracturing, unzipping of the lap splice and bond failure between the concrete and reinforcement in the previous reversed load cycle. No bar buckling was observed test specimen S02.
Photos showing the damage condition and state of specimens S01 and S02 at different performance points are shown in Figures 6.18 and 6.19 respectively. Figures 6.18(a) and 6.18(b) show the condition of S01 at peak lateral load and Figures 6.18(c) and 6.18(d) show its condition at lateral load failure (i.e. the lateral strength of the specimen dropped below 80% of its maximum capacity). Similarly, Figures 6.19(a) and 6.19(b) show the condition of S02 at peak lateral load and Figures 6.19(c) and 6.19(d) show its condition at lateral load failure. The photos of S02 show the web element of the core. Additional photos showing the damage progression of specimens S01 and S02 for the duration of the test are presented in Appendix C.

Figure 6.18: Photos of specimen S01 condition at peak strength and lateral load failure.

Figure 6.19: Photos of specimen S02 condition at peak strength and lateral load failure.
Prior to lateral load failure of specimen S02, the building core underwent a minor loss in its lateral strength equal to approximately 10% of its maximum capacity, occurring from a lateral drift value of 0.7% up to a value of 2.2% (refer Figure 6.17). This loss in lateral strength prior to lateral load failure is believed to be the result of a different mechanism than that which has been discussed in the previous paragraph. After the moulds were stripped during construction of the building core it was seen that some areas towards the base of wall needed to be patch fixed because poor vibration of the concrete was achieved in these locations. The gradual 10% loss in lateral capacity prior to the ‘real’ lateral load failure is believed to be due to local failure of these patch repaired sections of concrete near the base of the wall.

Both test specimens generally experienced minimal strength degradation between the first and second loading cycle for each respective displacement increment. This is illustrated in Figure 6.20, which shows the backbone force-displacement curve for each loading cycle. The peak displacement, drift and force values to construct the backbone curves in Figure 6.20 have been summarised and tabulated in Appendix C for the convenience of the reader.

![Figure 6.20: Force-displacement backbone curves highlighting strength degradation from load cycle 1 to 2 (left – test specimen S01; right – test specimen S02).](image)

The lap splice of the vertical reinforcement at the base of the wall resulted in atypical cracking to what is usually observed in RC wall testing. The length of the lap splice at the base of the wall was determined in accordance with AS 3600, which results in a conservative length of the splice that is at a minimum, always greater than two times the development length of the respective vertical reinforcement. The AS 3600 procedure calculates the bond length using a capacity reduction factor of 0.6 [X36] and then further stipulates the lap length shall be 1.25 times the bond length, i.e. \((1/0.6) \times 1.25 > 2\). This means there will be a region at the base of the wall having effectively double the amount of vertical reinforcement and hence a much larger moment capacity than the section of wall directly above and below. This creates a localised region of overstrength at the base of wall in the plastic hinge region with only hairline or partial incomplete cracks occurring. The inelastic plastic deformations are then concentrated above and below the lap splice region. This behaviour is illustrated in the curvature distributions for both test specimens, which are presented in Figures 6.21 to 6.24.
Test specimen S01 had a significantly longer lap length compared to test specimen S02 (900 mm compared to 500 mm), as this specimen was detailed with larger 20 mm nominal diameter bars compared to the 12 mm nominal diameter bars used in S02. This meant the localised region of overstrength in S01 much higher up the wall compared to S02. Both specimens were tested with the same shear-span ratio, meaning the applied moment at the top of the splice relative to the maximum moment at the bottom of the splice (i.e. the base of the wall) was smaller in S01 then S02. The applied moment at the top of splice was equal to 88% and 94% of the maximum moment at the base of the wall for specimens S01 and S02 respectively. The higher relative moment in S02 meant larger curvatures could be developed at the top of splice compared to S01. The location on the tension/compression flanges of test section S02 which were used to calculate the curvature and strain distributions in Figures 6.23 and 6.24 are shown in Figure 6.26.
Test specimen S02, in the negative direction loading cycles, developed significantly higher compressive strains in the lap splice region (i.e. Figure 6.24), compared to its positive direction loading cycles and both positive and negative directions in test specimen S01. This was possibly due to the concrete in this region of the wall being locally weaker from poor compaction, which easily could have resulted from the locally higher percentage of reinforcement across the lap splice region and the difficulty in pouring and vibrating concrete down a 2600 mm high wall that is only 130 mm thick. Irrespective, an interesting behaviour was observed where on one side of the wall, the plastic tensions strains were concentrated above the lap splice and then on the other side of the wall the plastic compression strains were concentrated across the lap splice region. This behaviour has been further illustrated in Figure 6.25.
Structural Engineers Association of California [X44] recommended lap splicing of reinforcement is avoided in plastic hinge regions of walls as they note “steel yielding and strains tend to concentrate over a short length of reinforcement at one or both ends of the lap splice length”, which then reduces the rotation and ductility capacity of the section. Concentration of yielding and plastic strains over short lengths was observed in test specimen S01 (Figures 6.21 and 6.22), further justifying this assertion. It should be noted though, that despite this, the specimen still exhibited a desirable ductile failure mode.

Lowes et al. [L18] performed an experimental study that included three ductile rectangular wall specimens with lap splices of the vertical reinforcement in the plastic hinge region. It was reported that the lap splice resulted in the damage being concentrated at the top and bottom of the splice, similar to what was observed in this testing. The lap splice region in these specimens was well confined and prevented any compression failure of concrete in this region, unlike what was observed in test specimen S02. Aaleti et al. [A4] performed an experimental study on three ductile rectangular wall specimens and included one specimen that was detailed with a lap splice at the base of the wall. Similar behaviour was also observed in this test, however significantly more damage occurred at the base of the wall compared to the top of the splice. This was due to the applied moment at the top of splice being 72% and 81% (a different length lap splice was provided at each end of the wall), of the maximum moment at the bottom of the splice and the base of the wall. Whereas the walls tested by Lowes et al. [L18] had much higher percentages, which were either 90% or 93% and allowed for the damage to occur at both the top and bottom of the splice, similar to this testing.

Spalling of the cover concrete on each corner of test specimen S02 occurred prior to any spalling or general concrete failure in any other region of the wall. This spalling was initiated by vertical cracking that began from +0.8%/-0.7% lateral drift (refer crack progression maps in Appendix C) and then further progressed and resulted in the cover concrete on the corners softening and effectively taking no load at +2.3%/-2.2% lateral drift. The extent of spalling around the corners is shown diagrammatically in Figure 6.26 and pictured in Figure 6.27.

The compressive strains would have been higher in the corners compared to the compressive flange generally, however this is not believed to be the root cause of the spalling and failure of this cover concrete. The vertical cracking and ensuing spalling extended up the height of the lap splice, where in the corner regions, there was effectively eight 12 mm bars across this height of the wall.
It is believed this created a highly congested region in the corners where the core concrete (i.e. the concrete inside these eight bars) was effectively cut off from the outside cover concrete. This then allowed the cover concrete around the corners, across the height of the lap splice, to break away.

![Diagram](image)

**Figure 6.26**: Corner spalling extents in test specimen S02.

(a) photo 1  (b) photo 2  (c) photo 3  (d) photo 4

Note: refer to Figure 6.26 for key plan showing the location each photo was taken from.

**Figure 6.27**: Corner spalling in test specimen S02 (cycle 97, -2.2% lateral drift).

Both specimens were detailed using moderate to high percentages of vertical reinforcement (1.8% and 1.4% for S01 and S02 respectively) and as such significant cracking was expected, since crack spacing is a function of the tensile strength of the concrete and the percentage of vertical reinforcement, refer Sections 3.5.4 and 5.6.1.1. The crack distribution of each specimen is presented in Figure 6.28 and shows significant distributing cracking occurred. The progression of cracking throughout the duration of the test was documented and is presented in Appendix C.

The process used for correcting the lateral displacement response of the specimens (as presented in Figure 6.13 and 6.16) for sliding and rigid body rotations (i.e. uplift of the foundation block) is presented in Appendix C.
6.4.2 Equivalent Single Degree-of-Freedom Response

The loading protocol for both test specimens consisted of applying a lateral displacement coupled with an in-plane lateral moment to simulate the response of a taller four-storey wall, as thoroughly discussed and described in the previous sections (e.g. Figure 6.3). This meant that the lateral in-plane displacements of each specimen reported (i.e. Figures 6.13 and 6.16) was the displacement at the top of the first storey (i.e. Figure 6.29(b)), as opposed to the effective displacement of the wall (i.e. Figure 6.29(c)). The effective displacement corresponds to the displacement of the equivalent single degree of freedom (i.e. 1DOF) system for the four-storey wall. A triangular lateral load distribution was assumed (i.e. Figure 6.29(a)), which resulted in a shear-span ratio of 6.5 and the equivalent 1DOF system having an effective height equal to three quarters of the overall height of the four-storey wall, i.e. $H_{eff} = 7.8m$. When assessing the performance of the wall, e.g. effective stiffness, displacement ductility or unpacking the different components of deformations, the effective response (i.e. equivalent single DOF system response), should be used, otherwise misleading results our outcomes could be reported, as shown in subsequent sub-sections.
To calculate the effective displacement (i.e. $\Delta_{eff}$), empirical relationships for flexure and shear displacement were determined for each specimen. The effective displacement was calculated by adding the measured displacement of the test specimen and the theoretical contribution of flexure and shear displacement from the top two thirds of the specimen, i.e. Equation 6.4.

$$\Delta_{eff} = \Delta_{test} + \Delta_f + \Delta_s$$ \hspace{1cm} … 6.4

Where:

- $\Delta_{test}$ = displacement of the test specimen
- $\Delta_f$ = flexure displacement contribution from the top two thirds of the theoretical specimen
- $\Delta_s$ = shear displacement contribution from the top two thirds of the theoretical specimen

The flexure displacement was calculated by firstly determining a theoretical average curvature distribution of the portion of wall between the top of the test specimen and the effective height for each discrete time interval of the test. The flexure displacement was then calculated by double integrating each of these theoretical curvature distributions for each discrete time interval.

To determine the average curvature distribution, the curvature across the top half of the specimen, outside the plastic hinge region where essentially elastic behaviour was present, was used to develop an empirical moment versus curvature relationship for each specimen (i.e. Figure 6.30). The theoretical length of wall above the test specimen was subdivided into 20 discrete lengths and the average curvature for each discrete segment was determined using the average bending moment for that segment and the relationship presented in Figure 6.30. This process allowed for an average curvature distribution for the theoretical top two thirds of the wall to be developed for each discrete time interval.

A similar process was used to calculate shear displacement, however an empirical effective shear modulus, i.e. $[GA]_{eff}$, versus moment relationship was developed for each specimen (i.e. Figure 6.31). The effective shear modulus relationship was determined using the top half of the specimen where essentially elastic response was observed. This relationship has much greater scatter than the curvature-moment relationship for each respective specimen (i.e. Figure 6.30 versus Figure 6.31); however, the shear displacement contribution to the specimen’s total displacement is much smaller than the flexure displacement contribution and as such, the variation has minimal effect on the final displacement.

The extra shear displacement contribution to the total effective displacement was calculated using Equation 6.5. The effective shear modulus for each of the 20 discrete segments, which were used for calculating the flexure component, was calculated using Figure 6.31. The shear displacement of each discrete segment could then be calculated and summed together using Equation 6.5.

$$\Delta_s = \sum_{i=1}^{n} \frac{F_{test} L_i}{[GA]_{eff,i}}$$ \hspace{1cm} … 6.5

Where:

- $L_i$ = length of the $i$-th segment
- $[GA]_{eff,i}$ = effective shear modulus of the $i$-th segment

The force-displacement response of the equivalent 1-DOF system for specimens S01 and S02, calculated using Equation 6.4, are presented in Figure 6.32.
Figure 6.30: Empirical curvature-moment relationship for test specimens S01 and S02.

Figure 6.31: Empirical shear modulus-moment relationship for test specimens S01 and S02.
Figure 6.32: 1-DOF force-displacement response of test specimen S01 (left) and S02 (right).

6.4.3 Deformation Components

The different components of deformation of each test specimen were determined using the V-STARS photogrammetry system. The flexural deformation was calculated by double-integrating the curvature distribution of the section (i.e. Figures 6.21 to 6.24), which was determined using the extreme axis tension and compression strains obtained from the photogrammetry measurements. Any sliding shear deformation was determined directly from the displacement profile of the wall, also obtained from the photogrammetry measurements. The shear deformation was then taken as the difference between the overall displacement and the combined flexural and sliding shear deformation, as shown in Figure 6.33. Additional displacement profiles are also provided in Appendix C.

Figure 6.33: Deformation components – example displacement profile of each test specimens.
The percentage of each deformation component (i.e. flexure, shear and sliding shear deformation), for each lateral displacement increment photogrammetry measurements are presented in Figure 6.34 for both test specimens S01 and S02. These deformations components are calculated based on the test specimen response (i.e. Figure 6.29(b)). The deformation components were recalculated for both S01 and S02 using the equivalent 1-DOF response (i.e. Figure 6.29(c)) and are presented in Figure 6.35. It is shown in both these figures that flexural deformations are the dominant component of displacement for both test specimens. This is expected, given both specimens were tested as very slender walls with a high shear-span ratio of 6.5. Despite the shear-span ratio being the same for both specimens, the building core specimen (i.e. S02) had a smaller overall component of flexural deformation than the rectangular wall (i.e. S01). This is expected since non-rectangular walls typically have larger components of shear deformation compared to rectangular walls of the same shear-span ratio, as shown in Section 4.1, Figure 4.2.

The percentage of flexural deformation in S01 and S02, based on the test specimen response (i.e. Figure 6.34), was 81% and 70% respectively. However, for the equivalent 1-DOF response (i.e. Figure 6.35), the percentage of flexural deformation increases to 93% and 89% respectively. Meaning the shear deformations are also negligible. Further, based on the test specimen response, it appears that the sliding shear deformations in S02 are quite significant. However, when this is observed in the larger context of the 1-DOF response, it is evident that the sliding shear is of little concern to the overall response. The percentages of flexure/shear deformation observed in this testing were consistent with the values reported by Massone and Wallace [M29] for the four wall specimens tested by Thomsen and Wallace [T5], which had a shear-span ratios of three.

Beyer, Dazio and Priestley [B8] reported that slender RC walls that are flexure controlled and have a stable shear-transfer mechanism, have an approximately constant ratio of flexure to shear displacement across the entire ductility range. This is somewhat in agreement with the results seen for test specimens S01 and S02, where from lateral drifts of about 0.5%, the percentage of flexural deformation stayed roughly the same.

### 6.4.4 Displacement Ductility, Overstrength and Force Reduction Factor

The force reduction factor (i.e. displacement ductility multiplied by overstrength) is an important parameter when undertaking force-based seismic design, such as the widely used equivalent static analysis procedure in AS 1170.4 (as outlined in Section 2.4.2). The displacement ductility, overstrength and force reduction factor of each test specimen was firstly calculated using the measured lateral response of the one-storey specimen, i.e. Figure 6.29(b), which is presented in Figure 6.13 and 6.16 for test specimens S01 and S02 respectively. These values were then recalculated using the equivalent 1-DOF response, i.e. Figure 6.29(c), which is presented in Figure 6.32 for both S01 and S02.

The displacement ductility was calculated with reference to Priestley et al. [P4], as it is widely considered a definitive text in the field. Priestley et al. [P4] propose that the yield moment be taken as the point corresponding to a compressive strain of 0.002 being developed in the extreme compressive fibre of the section or yielding of the section’s extreme tensile reinforcement, whichever occurs first. The theoretical yield moment of each specimen was calculated using the fibre-element analysis method described in Section 4.3.2, the material properties of each respective test specimen (i.e. Tables 6.2 and 6.3) and the criteria proposed by Priestley et al. [P4] mentioned above. The lateral force and displacement corresponding to the theoretical yield moment first being developed is denoted as the yield force (i.e. \( F_y \)) and notional yield displacement (i.e. \( \Delta_y \)) respectively.
Figure 6.34: Deformation components of test specimen response.

Figure 6.35: Deformation components of equivalent 1-DOF response.
A line was drawn from the origin through the point corresponding to the yield force and notional yield displacement and projected up to a horizontal line corresponding to the maximum lateral force (i.e. $F_{\text{max}}$) of the specimen to create a bilinear backbone response curve. The intersection of these two lines was taken as the yield displacement (i.e. $\Delta_y$) and the point prior to the lateral strength of the section dropping below 80% of the maximum was taken as the ultimate displacement (i.e. $\Delta_u$) of the specimen. These two lines formed the bilinear approximation for each specimen, which was used to calculate the displacement ductility, overstrength and force reduction factor. The displacement ductility is equal to the ultimate displacement divided by the yield displacement, i.e. $\mu = \Delta_u / \Delta_y$. The overstrength is equal to the maximum force capacity divided by the yield force, i.e. $\Omega = F_{\text{max}} / F_y$. The force reduction factor is equal to the overstrength multiplied by the displacement ductility, i.e. $R_f = \Omega \mu$.

These values and the bilinear approximation for specimens S01 and S02 are presented in Figure 6.36 for the test specimen response and Figure 6.37 for the equivalent 1-DOF response. The force and displacement values for each bilinear approximation are summarised in Table 6.6 and the overstrength, displacement ductility and force reductions factors are summarised in Table 6.7.

### Table 6.6: Test specimen S01 and S02 bilinear response.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$M_y$ (kNm)</th>
<th>$F_y$ (kN)</th>
<th>$\Delta_y$ (mm drift)</th>
<th>$F_{\text{max}}$ (kN)</th>
<th>$\Delta_y$ (mm drift)</th>
<th>$\Delta_u$ (mm drift)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01 (test specimen)</td>
<td>1021*</td>
<td>130.8</td>
<td>14.2 (0.5%)</td>
<td>171.8</td>
<td>18.7 (0.7%)</td>
<td>55.7 (2.1%)</td>
</tr>
<tr>
<td>S01 (1-DOF response)</td>
<td>72.3 (0.9%)</td>
<td>95.0 (1.2%)</td>
<td>198 (2.5%)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S02 (test specimen)</td>
<td>2291*</td>
<td>293.7</td>
<td>15.2 (0.6%)</td>
<td>316.6</td>
<td>16.4 (0.6%)</td>
<td>39.7 (1.5%)</td>
</tr>
<tr>
<td>S02 (1-DOF response)</td>
<td>63.1 (0.8%)</td>
<td>680 (0.9%)</td>
<td>130 (1.7%)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* For both specimens, the theoretical yield moment was governed by yielding of the extreme tensile reinforcement of the section.

### Table 6.7: Test specimen S01 and S02 force-based analysis coefficients.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\Omega$</th>
<th>$\mu$</th>
<th>$R_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01 (test specimen)</td>
<td>1.3</td>
<td>3.0</td>
<td>3.9</td>
</tr>
<tr>
<td>S01 (1-DOF response)</td>
<td>1.3</td>
<td>2.1</td>
<td>2.7</td>
</tr>
<tr>
<td>S02 (test specimen)</td>
<td>1.1</td>
<td>2.4</td>
<td>2.6</td>
</tr>
<tr>
<td>S02 (1-DOF response)</td>
<td>1.1</td>
<td>1.9</td>
<td>2.1</td>
</tr>
</tbody>
</table>

The test setup used in this experimental program where only the bottom portion of the wall is constructed has become a widely used testing procedure in recent years (e.g. [L8, L9, L18, M24, R1, S9, S13]), as it allows for large scale wall testing, with high shear-span ratios, to be undertaken that may otherwise be logistically not possible to perform. The different displacement ductility values and components of deformations calculated using the test specimen response and the equivalent 1-DOF response highlight the importance of ensuing that test data is correctly interpreted when undertaking complex test setups of this nature. Further, using the test specimen response to calculate these values results in larger ductility values and larger components of shear deformation.
Figure 6.36: Displacement ductility and overstrength of specimens S01 (left) and S02 (right), based on test specimen response.

Figure 6.37: Displacement ductility and overstrength of specimens S01 (left) and S02 (right), based on equivalent 1-DOF response.

The displacement ductility was determined to be 2.1 and 1.9 respectively S01 and S02. Similarly, the overstrength was determined to be 1.3 and 1.1 respectively and the force reduction factors were determined to be 2.7 and 2.1 respectively, as summarised in Table 6.7. Both specimens were detailed in accordance with the main body of AS 3600, which allows a limited ductile classification to be adopted. Displacement ductility and overstrength factors of 2 and 1.3 respectively can be adopted for limited ductile RC structures when performing seismic analysis in accordance with AS 1170.4. This results in a force reduction factor of 2.6. Both specimens achieved displacement ductility factors approximately equal to what the code, which suggests the current value 2.0 is appropriate. Test specimen S01 had an overstrength value of 1.3, which results in a force reduction value of 2.7, just higher than the code value of 2.6.
Test specimen S02 had a lower amount of overstrength to that which is assumed in the code, i.e. 1.1 compared to 1.3, and resulted in a force reduction value of 2.1, somewhat lower than what is allowed in AS 1107.4 and AS 3600. It is hypothesised that if the specimen did not have a lower concrete compressive strength than what was specified (i.e. 31.6MPa compared to 40MPa) nor required the patch fixing discussed in the previous section, the specimen would have seen a higher post yield strength gain, which would have resulted in a higher overstrength value, higher ultimate displacement and higher displacement ductility, likely resulting in better performance than what is assumed in the code, i.e. the force reduction factor would likely have been greater than 2.6. This is discussed further in Section 8.2.2.1.

It should be noted that comparing the overstrength, displacement ductility and force reduction factors from AS 1170.4 to the results of test specimens S01 and S02 can be somewhat misleading since these factors for these specimens are calculated using their actual yield displacements. Whereas when force-based analysis is performed using AS 1170.4 the yield displacement of the building or structure is essentially controlled by the empirical equation used for calculating the first mode natural period (i.e. Equation 2.7, refer Section 2.4.2), which is a very conservative equation. Therefore, AS 1170.4 underestimates the yield displacement (as a result of the conservative natural period calculation), which means higher force reduction factors would be required to factor up the smaller yield displacement to achieve the equivalent ultimate displacement.

6.4.5 Effective Stiffness

The effective stiffness of the specimens, has been assumed to be the slope of a line on the force-displacement graph from the origin through the point which corresponds to first yield moment of the cross section. For test specimens S01 and S02 this resulted in an effective stiffness of 1809 and 4655 kN/m respectively, when using the first yield displacement and force values for the equivalent 1-DOF response presented in Figure 6.37 and summarised in Table 6.6.

The effective moment of inertia (i.e. $I_{eff}$) of each specimen can then be calculating using Equation 6.6, which is the formula for calculating the stiffness of a cantilever flexural element. In this scenario, the length is taken as the effective height. The elastic modulus of concrete (i.e. $E_c$) is determined using the equation from AS 3600 and the concrete compressive strength of each specimen, which for test specimen S01 and S02 are calculated to be approximately 31,000 and 28,400 MPa respectively. The effective height (i.e. $H_{eff}$) is 7.8 m. The effective and uncracked (i.e. gross) moment of inertia of each specimen have been summarised in Table 6.8.

$$k_{eff} = \frac{3E_c I_{eff}}{H_{eff}^3} \quad \ldots 6.6$$

**Table 6.8:** Effective moment of inertia.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$I_{eff}$</th>
<th>$I_g$</th>
<th>$\frac{I_{eff}}{I_g}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>0.0092 m$^4$</td>
<td>0.0288 m$^4$</td>
<td>0.32</td>
</tr>
<tr>
<td>S02</td>
<td>0.0259 m$^4$</td>
<td>0.1064 m$^4$</td>
<td>0.24</td>
</tr>
</tbody>
</table>
The effective moment of inertia for specimens S01 and S02 was 0.32 and 0.24 respectively. These values are relatively low, compared to many popular models in literature, as shown in Table 6.9, which compares the results of the test specimens to the models proposed by Adebar et al. [A3], Elwood and Eberhard [E6], Fenwick and Bull [F4], Fenwick, Hunt and Bull [F5], NZS 3101:Part 2 [X38] and Paulay and Priestley [P1]. The Fenwick and Bull [F4] model predicts the stiffness of the rectangular wall specimen (i.e. S01) fairly well and the Paulay and Priestley [P1] model predicts the stiffness of the building core specimen (i.e. S02) fairly well also. Otherwise, correlation between the test specimens and the models is not very good.

Table 6.9: Comparison of effective moment of inertia with literature.

<table>
<thead>
<tr>
<th></th>
<th>Adebar et al.</th>
<th>Elwood &amp; Eberhard</th>
<th>Fenwick &amp; Bull</th>
<th>Fenwick et al.</th>
<th>NZS 3101 Part 2</th>
<th>Paulay &amp; Priestley</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_{\text{eff}}/I_g$ predicted S01</td>
<td>0.36</td>
<td>0.48</td>
<td>0.32</td>
<td>0.38</td>
<td>0.37</td>
<td>0.25</td>
</tr>
<tr>
<td>$I_{\text{eff}}/I_g$ predicted S02</td>
<td>0.39</td>
<td>0.55</td>
<td>0.32</td>
<td>0.39</td>
<td>0.38</td>
<td>0.26</td>
</tr>
<tr>
<td>$I_{\text{eff}}/I_g$ actual S01</td>
<td>113%</td>
<td>149%</td>
<td>99%</td>
<td>117%</td>
<td>116%</td>
<td>79%</td>
</tr>
<tr>
<td>$I_{\text{eff}}/I_g$ actual S02</td>
<td>164%</td>
<td>229%</td>
<td>133%</td>
<td>161%</td>
<td>159%</td>
<td>109%</td>
</tr>
</tbody>
</table>

### 6.5 Plastic Hinge Development in Limited Ductile RC Walls

The post yield deformation response was somewhat different to the response commonly seen in RC wall testing performed generally in literature. Typically – when no lap splice is present at the base of the wall – the wall either develops a traditional plastic hinge with distributed cracking at the base of the wall, where the inelastic plastic behaviour is ‘spread’ across multiple cracks, or when the percentage of vertical reinforcement is not sufficient to initiate distributing cracking, a single crack forms with a concentration of the inelastic plastic behaviour in one location (i.e. Figures 6.38(a) and 6.38(b) respectively). The latter of these two scenarios, which has received much research attention in recent years (e.g. [H4, L8, L9, L10]), has a significantly reduced inelastic displacement capacity compared to the former and is generally associated with the scenario where the cracking moment capacity of the wall is greater than the ultimate moment capacity of the wall.

It has been shown in this testing that neither of the two aforementioned post-yield plastic hinge models are developed. The lap splice at the base of the wall – which is common practice in Australia and generally associated with limited ductile RC wall detailing – results in a region at the base of the wall having effective double the amount of vertical reinforcement and hence a much larger moment capacity than the section of wall directly above and below. This results in a region of overstrength at the base of the wall where only hairline cracks develop. This behaviour leads to the development of either a ‘two-crack’ plastic hinge model or a single crack plus a shifted traditional plastic hinge model (i.e. Figures 6.38(a) and 6.38(b) respectively).
The two-crack plastic model where only two predominate cracks develop, one at the bottom of the lap splice and one at the top of the lap splice, with the majority of the plastic behaviour concentrated in these two locations. The shifted plastic hinge model is where a traditional plastic hinge (i.e. Figure 6.38(a)) develops at the top of the lap splice, in addition to a single large crack forming at the base of the splice. The former latter responses will be dictated by the ratio of the applied moment at the base of the wall to the applied moment at the top of the lap splice, which is in turn dependent on the shear-span ratio of the wall (i.e. slenderness) and length of the splice.

\[ \frac{M_{base}}{M_{top}} \]

Figure 6.38: Plastic hinge development in RC walls.

The curvature distributions of the rectangular wall specimen (i.e. S01) in Figures 6.21 and 6.22 show the two-crack plastic hinge model was formed in the wall (i.e. Type C Figure 6.38(c)). The curvature distributions of the building core specimen (i.e. S02) in Figures 6.23 and 6.24 show the shifted plastic model was formed in the wall (i.e. Type D Figure 6.38(d)). The bending moment at the top of the splice was 88% and 94% of the maximum moment at the base of the wall for test specimen S01 and S02 respectively. The higher relative bending moment at the top of the splice in S02 initiated the Type D behaviour. Whereas the slightly lower relative bending moment at the splice in S01 meant that the Type C behaviour was developed with higher concentrations of plasticity.
6.6 Conclusions

This chapter has presented the details and results of the second experimental testing program. Two specimens were constructed and tested under in-plane unidirectional cyclic lateral load with a shear-span ratio of 6.5. The first specimen was a rectangular wall with a vertical reinforcement ratio of 1.8% and an axial load ratio of 6.5%. The second specimen was a building core with a box-shaped cross section, a vertical reinforcement ratio of 1.4% and an axial load ratio of 7.7%. Both specimens were constructed with lap splices of the vertical reinforcement at the base of the wall in the plastic hinge region.

The rectangular wall and building core both achieved a displacement ductility of about 2 before serious strength degradation started to occur, which is in good agreement with the ductility assumptions usually adopted by Australian designers when using the Australian earthquake loading standard, AS 1170.4. The ultimate failure mechanism of the rectangular wall was crushing of the concrete in the extreme compressive fibre of the wall, whereas the building core specimen failed due to the development of high tensile strains in the vertical reinforcement, which resulted in a combination of fracturing of the vertical reinforcement, unzipping of the lap splice and degradation of the concrete due to bond failure between the concrete and reinforcement.

The rectangular wall (i.e. S01) and building core (i.e. S02) specimens were able to achieve (at the top of the specimen) ±2.1% and ±1.5% lateral drift respectively prior to lateral load failure of the specimens occurring (i.e. the lateral strength dropped below 80% of the respective maximum capacity). The walls continued to achieve ±4.2% and ±4.5% lateral drift respectively prior to axial load failure occurring (i.e. complete structural collapse). The test specimen response was converted to an equivalent 1-DOF response and the lateral drift associated with lateral load failure was increased to 2.5% and 1.7% for specimens S01 and S02 respectively. The response of the specimens predominantly consisted of flexural deformations with minimal shear deformations. The flexural deformations were calculated to be approximately 90% of the total displacement.

The test results of both specimens showed that a traditional plastic hinge with distributed cracking and distributed plasticity, as commonly seen in RC wall testing, was not achieved due to the lap splice at the base of the wall. The lap splice created a region of overstrength, over which only hairline cracks formed with major cracks either side, i.e. at the base of the wall and the top of the lap splice. The plastic rotation and curvature of the wall was concentrated within these two locations.
Chapter 7

Experimental Testing of Jointed Precast Walls and Connections

This chapter presents an overview, results and discussion on the third program of large-scale experimental testing performed on RC walls. The third program of experimental work consisted of three jointed precast building core tests (i.e. system level tests) and three precast building core connection tests (i.e. component level tests). The three building core specimens were constructed using industry standard construction methods, reinforcement detailing and panel connection details (as documented in Chapter 3). The building cores consisted of four individual rectangular panels that were arranged to form a box shaped building core. Each adjacent panel was joined together using industry standard welded stitch plate connections and then connected to foundation blocks on the top and bottom using industry standard grout tube connections. The component level panel connection tests consisted of an initial baseline specimen, which consisted of typical welded stitch plate connection, and two new types of innovative prototype panel connection specimens. The primary goal was to develop a panel connection that did not require any site welding, yet had a higher strength capacity and stiffness than the standard stitch plate connections. The testing was performed using the MAST System in the Smart Structures Laboratory at Swinburne University of Technology.

7.1 Precast Wall Testing Objectives and Methodology

7.1.1 Background and Objectives

In recent years precast concrete walls have become increasingly popular in Australia, particularly in the south-eastern states. Precast construction offers many benefits over traditional cast in-situ RC construction, which includes more efficient and faster on-site construction timeframes and high-quality construction tolerances that can be achieved in an off-site warehouse manufacturing environment.
Despite the widespread adoption of precast walls and building cores, no experimental studies have been performed to assess their actual performance. The performance of such systems has typically been assessed and determined using theoretical approaches in structural engineering design offices.

Precast walls are often either used as individual rectangular walls, with or without openings, around the perimeter of a building or assembled together to form jointed building cores. This has been illustrated in Figure 7.1, which shows an example floor plan from a typical multi-storey residential building in Melbourne. Precast walls are typically joined to foundations or walls above/below using grout tube connections and horizontally to adjacent panels using 'welded stitch plate' (WSP) connections (as shown in Figure 7.2). The grout tube connection has a corrugated grout tube cast into the panel, typically about twice the diameter of the starter bar or dowel bar that slots into it, allowing for a generous amount of construction tolerance so the panels can be easily erected and accurately aligned on site. After the panels are erected on site, the base of the panel is 'dry packed' with a cementitious grout. The grout tube is usually then filled the next day using a flowable high strength cementitious grout. This creates an integral connection between the base of the panel and the surrounding RC structure capable of transferring the required design forces, i.e. tension and compression axial forces and in-plane and out-of-plane shear forces.

---

**Figure 7.1:** Floor plan of a typical multi-storey residential building with precast walls.
The WSP connection has steel plates, with shear studs welded to the rear side, cast into the edges of two adjacent panels. After the panels are erected on site, a third steel plate is site welded to each adjacent cast in plate. This connection also allows a generous amount of construction tolerance so the panels can be easily and accurately aligned into their required position on site. The WSP connections are required to transfer vertical shear forces between adjacent panels to allow the individual panels to work together to form one overall composite cross section, i.e. building core, as illustrated in Figure 7.3. The construction process for a precast building core is illustrated in Appendix D, which includes time lapse photos showing the construction and assembly process for test specimen S05.

Horizontal panel-to-panel connections have two primary performance metrics that need to be met, such that composite action between adjacent panels can be effectively achieved in precast construction. These are strength and stiffness. If the connection does not have sufficient strength, it will fail prematurely, hindering any composite behaviour between the panels. This performance metric is easily dealt with in a design office, as force-based design procedures are generally well developed and well understood, but often with varying interpretations.
This was highlighted by a blind study into the design capacity of these connections (i.e. welded stitch plates), which was performed with designers from industry. The blind study showed there was varying opinion as to how the design of these ‘standard’ connections should be performed and resulted in a relatively wide range of capacity values for the same connection. This blind study is documented later in this Chapter in Section 7.3.5.

The second metric, i.e. stiffness, is much harder to assess in a design office and ideally requires experimental test data. The stiffness of the connection is often overlooked and then it is blindly assumed that the connection is essentially ‘infinitely’ stiff. The stiffness of the connection is important as it will ultimately decide the amount of composite behaviour that can be developed, assuming there is sufficient strength. If the connection has high strength, but is very flexible, it will be ineffective at allowing composite behaviour to develop and the panels will act independently (i.e. the top stress diagram in Figure 7.3). However, as the connection becomes stiffer and approaches ‘infinitely’ stiff; full composite action of the panels will be developed (i.e. the bottom stress diagram in Figure 7.3).

Currently the literature provides little to no guidance as to what amount of connection stiffness is required to develop full or partial composite behaviour between panels. Similarly, very little guidance is provided as to what the stiffness of a WSP connection is, despite the widespread adoption and use in hundreds, most likely thousands, of buildings across Australia. The precast testing program performed and documented in this Chapter was primarily implemented in an effort to address this gap in the literature.

**Figure 7.3:** Composite behaviour of a jointed precast building core.
The secondary objective of the precast testing program was to develop new prototype connections for precast building cores. This aspect of the testing program was driven by industry precasters in Melbourne in an effort to ‘move away’ from WSP connections, which have become the pseudo industry standard for precast connections in Australia. Welded stitch plate connections are both expensive and slow the construction cycle times, largely due to the requirement to bring an additional trade on site that is otherwise not required for any other activities during the RC construction phase of a project. As such the primary requirement for a new prototype connection was that it did not require any welding or the use of other specialist sub-contractors.

The secondary requirements were that the connection was stiffer and stronger than the equivalent WSP connection. A stiffer connection is desirable as it would allow for more effective composite action to be developed (as discussed previously). A stronger connection is desirable as it would allow a larger ‘design margin’ before wet joints are required. Wet joints are where a corner segment between adjacent precast panels is poured in-situ on site, as discussed in Section 3.3.2 and illustrated in Figure 3.11. Well designed and constructed wet joints essentially allow a jointed precast building core to behave identically to a traditional cast in-situ building core, however come at quite a significant economic cost. Wet joints increase the overall construction cost of precast building cores significantly and it has been anecdotally suggested by various contractors that once wet joints are required, the cost difference between a jointed precast building core and a traditional cast in-situ building core is marginal in many buildings.

7.1.2 Methodology and Testing Plan

The precast building core experimental testing program consisted of three large scale system tests and three large scale component level tests. The system level tests (i.e. S03, S04 and S05) were precast equivalents of the cast in-situ building core specimen documented in Chapter 6. The precast building cores consisted four individual rectangular panels that were connected vertically using grout tube connections and horizontally using WSP connections (as shown in Figure 7.2). Similar to the cast in-situ building core specimen, i.e. test specimen S02, the precast building core specimens were a one-storey test specimen that was meant to represent the ground floor component of a taller four-storey building core (refer Figure 6.3). The component level tests (i.e. J01, J02 and J03) were a series of experimental tests to assess the vertical shear capacity of the horizontal panel-to-panel connections. The difference between the system level and component level tests is shown in Figure 7.4.

The first of the three component level connection specimens (i.e. test specimen J01) was the baseline test, which was a WSP connection and a replica of the connection used to construct the third precast building core specimen (i.e. test specimen S05). The second and third component level specimens (i.e. test specimen J02 and J03 respectively) were constructed using two new prototype connections developed for precast construction to meet the aims and objectives discussed previously. J02 was developed as a direct replacement for WSP connections that would be simple to construct and not require any site welding. In Contrast, J03, was developed as a significantly stronger and stiffer alternative to stitch plates, however the trade-off being that is it more difficult to construct. Both J02 and J03 do not require any site welding to be performed. The preliminary prototype connection drawings for J02 and J03 are shown in Figure 7.5.

The system level building core tests and the component level connections tests are documented in the subsequent two sections of this Chapter, Sections 7.2 and 7.3 respectively. The chapter is concluded with proposed precast panel connection details, which are presented in Section 7.4.
**Figure 7.4:** Precast building core experimental testing methodology.

**Figure 7.5:** Preliminary sketches of prototype connections for precast building cores.
7.2 Precast Building Core Testing – Test Specimens S03 to S05

This section outlines test specimen details, instrumentation, test setup, loading protocol and results of the system level precast building core specimens.

7.2.1 Test Specimen Overview

The experimental system level precast building core testing program consisted of three specimens denoted S03, S04 and S05. The overall geometry of the three specimens is shown in Figures 7.6 and 7.7. The three specimens were constructed to generally match the geometry (i.e. height and wall length) of the cast in-situ wall testing presented in Chapter 6 and similarly were also one-storey elements meant to represent the ground floor component of a taller four-storey building core (refer Figure 6.3). This meant the three building core specimens were similarly tested with a shear-span ratio (i.e. $M^*/(V'L_w)$) of 6.5. To achieve the same bending moment and shear force response in the one-storey test specimen to that of the ground floor of a four-storey core, an in-plane lateral force and moment are applied to the specimen. The specimens have an inter-storey height of 2600 mm and if a triangular lateral load distribution is assumed, the applied moment is then equal to 5.2 times the applied lateral force (refer Equations 6.1 to 6.3).

The precast panels for all three specimens were manufactured by a local precast manufacturer in Melbourne. Test specimens S03 and S04 were assembled by the manufacturer, whereas the panels for test specimen S05 were delivered to the Smart Structures Laboratory (SSL) at Swinburne and the building core was assembled on site. Time lapse photos showing the assembly procedure of S05 are presented in Appendix D. Test specimens S03 and S04 were constructed using 130 mm thick precast panels, which matches the wall thickness of cast in-situ core (i.e. S02), while S05 was constructed using 150 mm thick precast panels.

![specimen S03 and S04 3D view](image)

![specimen S05 3D view](image)

**Figure 7.6:** Test specimens S03 to S05 perspective views.
The first two specimens (i.e. S03 and S04) were designed to be jointed precast concrete replicas of the cast in-situ building core specimen (i.e. S02) presented in Chapter 6. The first specimen (i.e. S03) was to have the same reinforcement content and rebar grade as S02 and the second specimen (i.e. S04) was to have the same reinforcement content as S02, however it would be constructed using low ductility rebar (i.e. D500L mesh) instead of normal ductility rebar (i.e. D500N bars). Due to some unfortunate construction errors by the precast contractor, the specimens ended up varying from what was originally intended, meaning neither S03 or S04 ended up being an exact replica of S02 and hence not allowing a direct point of comparison. These errors resulted in S03 having an unsymmetrical reinforcement layout and S04 having a very high content of vertical reinforcement, which was significantly higher than the associated dowel bar reinforcement in the grout tube connections. As a result, the precast contractor agreed to manufacture an additional third specimen, i.e. S05, which generally has the reinforcement layout S04 was intended to have.

In addition to the construction errors discussed above, some of the dowel bars in the bottom boundary element of specimens S03 and S04 were cast in the wrong location. This meant these bars (which are highlighted later in Figure 7.8) had to be cut off so new bars could be epoxied into the bottom boundary element in the correct locations. These bars were not epoxied in as specified and, during testing of the specimen, pull out failure of the bars, due to bond failure of the epoxy, occurred relatively early when low amounts of lateral load were being applied.

Despite the issues discussed above with respect to S03 and S04, which were discovered post testing, the results are still meaningful with the ‘as-built’ details and provided the opportunity for an additional third specimen (i.e. S05) to be designed, constructed and tested as means of compensation by the precast contractor. Specimen S05 will be the particular focus in the results section, Section 7.2.4.

The specimens were constructed using a standard N40 grade concrete mix, which has a minimum characteristic 28-day compressive cylinder strength of 40 MPa. The panels for each specimen were cast on different days, resulting in the actual concrete strengths for the individual panels used for each specimen being different. The actual concrete strengths on test day for the individual panels are summarised in Table 7.1. The panel numbering for each specimen used in Table 7.1 is presented in Figure 7.7.

![specimen S03 and S04 cross section](image1)

![specimen S05 cross section](image2)

**Figure 7.7:** Test specimens S03 to S05 cross sections.
Table 7.1: Concrete strength (precast wall system level tests).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specified strength</th>
<th>Actual strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Panel A</td>
</tr>
<tr>
<td>S03</td>
<td>40 MPa</td>
<td>47.9 MPa</td>
</tr>
<tr>
<td>S04</td>
<td>40 MPa</td>
<td>44.3 MPa</td>
</tr>
<tr>
<td>S05</td>
<td>40 MPa</td>
<td>44.5 MPa</td>
</tr>
</tbody>
</table>

The panel reinforcement and dowel reinforcement for specimens S03 to S05 is presented in Figure 7.8 and Tables 7.2 to 7.4. The panel numbering for each specimen used in Tables 7.2 to 7.4 is presented in Figure 7.7. The construction drawings for each test specimen are provided in Appendix A.

The pull-out failure of the two starter bars in each respective flange panel of specimens S03 and S04, which was discussed above and is shown in Figure 7.8, resulted in the specimens having effectively 50% less area of dowel bar reinforcement in the flanges. This meant the reinforcement ratio in the flanges reduced from 1.7% to 1.2%. This created a region of weakness in both S03 and S04 at the base of the panels, causing all the inelastic plastic behaviour to be concentrated in one location at the base of the core. The additional reinforcement that was accidentally included in S04 further exacerbated this problem. This is shown and discussed further in the results section.

SL82 and SL92 (as specified in Figure 7.8) denotes square welded wire mesh with L7.6 and L8.6 bars at 200 mm centres in each direction respectively. RL108 and RL1118 (as specified in Figure 7.8) denotes rectangular welded wire mesh, which has longitudinal and crosswise bars at 200 mm and 100 mm centres respectively. RL108 has L9.5 and L7.6 longitudinal and crosswise bars respectively, whereas RL1118 has L10.7 and L7.6 bars respectively.

Table 7.2: Panel reinforcement details.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Vertical reinforcement per face</th>
<th>Horizontal reinforcement per face</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Panel A</td>
<td>Panel B</td>
</tr>
<tr>
<td></td>
<td>Panel A</td>
<td>Panel B</td>
</tr>
<tr>
<td></td>
<td>Panel A</td>
<td>Panel B</td>
</tr>
<tr>
<td></td>
<td>Panel A</td>
<td>Panel B</td>
</tr>
<tr>
<td>S03</td>
<td>8-N12 and 6-L7.6</td>
<td>10-N12 and 6-L8.6</td>
</tr>
<tr>
<td></td>
<td>12-N12 and 11-L10.7</td>
<td>8-N12 and 9-L10.7</td>
</tr>
<tr>
<td></td>
<td>2-N12 and 11-L9.5</td>
<td>2-N12 and 8-L9.5</td>
</tr>
<tr>
<td>S04</td>
<td></td>
<td>L7.6 at 200 centres</td>
</tr>
<tr>
<td>S05</td>
<td></td>
<td>L7.6 at 200 centres</td>
</tr>
<tr>
<td></td>
<td></td>
<td>L7.6 at 200 centres</td>
</tr>
</tbody>
</table>

Table 7.3: Panel reinforcement ratios.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Vertical reinforcement ratio</th>
<th>Horizontal reinforcement ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Panel A</td>
<td>Panel B</td>
</tr>
<tr>
<td></td>
<td>Panel A</td>
<td>Panel B</td>
</tr>
<tr>
<td></td>
<td>Panel A</td>
<td>Panel B</td>
</tr>
<tr>
<td>S03</td>
<td>0.015</td>
<td>0.019</td>
</tr>
<tr>
<td>S04</td>
<td>0.030</td>
<td>0.030</td>
</tr>
<tr>
<td>S05</td>
<td>0.011</td>
<td>0.011</td>
</tr>
</tbody>
</table>
Figure 7.8: Test specimens S03 to S05 reinforcement plans.
### Table 7.4: Dowel reinforcement.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Dowel reinforcement</th>
<th>Dowel reinforcement ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Panel A</td>
<td>Panel B</td>
</tr>
<tr>
<td>S03</td>
<td>6-N24</td>
<td>6-N24</td>
</tr>
<tr>
<td>S04</td>
<td>6-N24</td>
<td>6-N24</td>
</tr>
<tr>
<td>S05</td>
<td>6-N24</td>
<td>6-N24</td>
</tr>
</tbody>
</table>

* The reinforcement ratio for panels A and B of specimens S03 and S04 was effectively decreased to 0.012 because two of the N24 dowel bars needed to be relocated (as shown in Figure 7.8) and when the new bars were epoxied into the boundary element block they were not epoxied correctly, allowing them to prematurely to pull-out and failure relatively early in the test.

† Similar to panels A and B as discussed above, one of the dowel bars in panel C of specimen S03 prematurely failed meaning the reinforcement ratio was effectively decreased to 0.011.

The specimens were detailed using D500N reinforcing bars and D500L welded wire mesh in accordance with AS/NZS 4671 [X29], which have a minimum characteristic yield stress of 500 MPa, strain hardening ratios of 1.08 and 1.03 respectively and ultimate strains of 5% and 1.5% respectively. N12 and L7.6 denotes 12 mm and 7.6 mm nominal diameter grade D500N and D500L reinforcing bars respectively. Samples from the reinforcement used for each specimen were tested to determine the mean material properties of the reinforcement (Table 7.5). The number of samples performed for each diameter and type or rebar varied from two to eight.

### Table 7.5: Reinforcement properties (precast wall system level tests).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement</th>
<th>$f_{sy}$</th>
<th>$f_{su}$</th>
<th>$f_{sy}/f_{su}$</th>
<th>$\varepsilon_{su}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S03/S04</td>
<td>N12 vertical bars</td>
<td>569 MPa</td>
<td>644 MPa</td>
<td>1.13</td>
<td>8.4%</td>
</tr>
<tr>
<td>S03/S04</td>
<td>N24 dowel bars</td>
<td>549 MPa</td>
<td>645 MPa</td>
<td>1.18</td>
<td>11.0%</td>
</tr>
<tr>
<td>S03</td>
<td>L8.6 vertical and horizontal bars</td>
<td>562 MPa</td>
<td>605 MPa</td>
<td>1.08</td>
<td>3.3%</td>
</tr>
<tr>
<td>S04</td>
<td>L10.7 vertical bars</td>
<td>678 MPa</td>
<td>776 MPa</td>
<td>1.14</td>
<td>3.5%</td>
</tr>
<tr>
<td>S04</td>
<td>L7.6 horizontal bars</td>
<td>632 MPa</td>
<td>719 MPa</td>
<td>1.14</td>
<td>5.0%</td>
</tr>
<tr>
<td>S05</td>
<td>N12 vertical bars</td>
<td>527 MPa</td>
<td>611 MPa</td>
<td>1.16</td>
<td>7.9%</td>
</tr>
<tr>
<td>S05</td>
<td>N24 dowel bars</td>
<td>503 MPa</td>
<td>609 MPa</td>
<td>1.21</td>
<td>12.2%</td>
</tr>
<tr>
<td>S05</td>
<td>L9.5 vertical bars</td>
<td>586 MPa</td>
<td>633 MPa</td>
<td>1.08</td>
<td>3.1%</td>
</tr>
</tbody>
</table>

The panels for specimens S03 to S05 were connected together using WSP connections, as shown in Figure 7.9. Test specimens S03 and S04 had ‘inside’ fixed stitch plates and S05 had ‘outside’ fixed stitch plates. The locations of the cast in plates for the stitch plate connections are shown in Figure 7.10. The cast in plates were grade 250 to AS/NZS 3678 [X45], which has a minimum characteristic yield stress of 260 MPa for 12 mm thick elements. The stitch plates were equal angle sections for S03 and S04 and flat bar sections for S05. The equal angle and flat bar sections were both grade 300 to AS/NZS 3679.1 [X46], which has a minimum characteristic yield stress of 300 MPa for 16 mm thick elements. Material samples of the structural steel plates and or sections for the stitch plate connections were not taken, however the steel plates and welds did not undergo any inelastic behaviour during the testing, maintaining an elastic response.
The panels were connected to the top and bottom boundary elements using a grout tube connection, as shown in Figure 7.11. The grout tube connection consisted of a 50 mm diameter corrugated grout tube cast into the top and bottom of the panel. After the panels were erected the grout tubes were gravity poured using the Aitken Freemans Tecgrout HS, which has a 90 MPa characteristic compressive strength at 28 days. The grout was mixed using the higher water content required to achieve a flowable mix – such that pouring of the grout tubes could be achieved without any air pockets forming – which meant the strength of the grout was lower than the characteristic strength. The grout strength varied between 67 and 78 MPa on test day. The grout strength was determined using 50 mm cube samples. This meant the grout strength was about 20 to 30 MPa stronger than the panels.

The grout tube connection had N24 dowel bars with an embedment length of 800 mm. The embedment length of the dowels was calculated generally with respect to the requirements in AS 3600 [X6] for calculating the development lap and lap length of deformed reinforcing bars in concrete. AS 3600 does not provide any guidance for calculating the embedment length for dowel bars in grout tube connections. A further discussion on this topic is provided in Chapter 5.
7.2.2 Instrumentation

The instrumentation setup for test specimens S03 to S05 was essentially the same as what was used for test specimens S01 and S02, which was presented in Section 6.2. This consisted of a combination of physical instrumentation attached to the test specimens and a contactless photogrammetry system. The physical instrumentation was primarily used to measure the overall global behaviour (e.g. in-plane displacements, rotations and axial behaviour) and the photogrammetry system was primarily used for quantifying the different types of deformations (e.g. flexure, shear and sliding) and sectional responses (e.g. strain and curvature distributions) of the specimens.

The photogrammetry system used was the V-STARS N series by Geodetic Systems. This system is a turnkey single camera photogrammetry system, which can be used to make discrete measurements of the test specimen while the testing procedure is paused. A complete discussion regarding the photogrammetry system and the heavy reliance on contactless measuring systems, as adopted in this testing, is presented in Section 6.2. Photogrammetry measurements were taken for each specimen on the second positive and negative loading cycle for each lateral displacement increment. The location of photogrammetry targets, where the x-y-z movement was calculated and recorded at discrete moments throughout the test, for each specimen, is shown in Figures 7.12 to 7.15.

A series of string potentiometers (SPOTs) were used to measure the in-plane lateral displacement, rotation and axial behaviour of the test specimens (refer Figure 6.6 for specimens S03 and S04 and refer Figure 7.16 for specimen S05). A series of laser displacement sensors were used to monitor and measure any slip or uplift (e.g. base rotation) during the testing of specimens S03 and S04 (Figure 6.6). However, for test specimen S05, linear potentiometers (LPOTs) were used in lieu of the laser displacement sensors for measuring the base slip and rotation (Figure 7.16). Additional string potentiometers were used during the testing of S05 only, to monitor and capture any shear sliding behaviour at the base of the panels (as shown in Figure 7.16). Whereas for test specimens S03 and S04, the photogrammetry system was used to monitor and capture any sliding shear behaviour.
Figure 7.12: Test specimens S03 to S05 photogrammetry key plans.

Figure 7.13: Test specimens S03 to S05 photogrammetry targets (approximate locations).
Figure 7.14: Test specimens S03 to S05 photogrammetry key plans.

Figure 7.15: Test specimen S05 photogrammetry targets (approximate locations).
A series of LVDTs, stacked vertically at each end of the wall, were used to verify the strain and curvature distributions determined from the photogrammetry system (e.g. Figures 6.6, 6.4 and 7.16). The LVDT stacks were used for all three specimens, i.e. S03 to S05. A summary table of all the physical instrumentation, including transducer details such as stroke length, is provided in Appendix D for specimen S05. The physical instrumentation for specimens S03 and S04 was the same as specimens S01 and S02 and a summary table for the latter is presented in Appendix C.

![Specimen S05 elevation A: instrumentation L01 to L20](image)

**Figure 7.16:** Test specimen S05 physical instrumentation.

### 7.2.3 Test Setup and Loading Protocol

Test specimens S03 to S05 were tested using the Multi-Axis Substructure Testing (MAST) System in the Smart Structures Laboratory (SSL) at Swinburne University of Technology. The MAST System is a state-of-the-art test machine capable of applying full six degree-of-freedom (DOF) loading in mixed-mode, switch-mode, hybrid or combination therein [H11]. The specifics of the MAST System and the test specimen setup are discussed in detail in Section 6.3.1.
The specimens were tested under unidirectional quasi-static cyclic test conditions matching what was performed for test specimens S01 and S02. A brief overview of the loading protocol for test specimens S03 to S05 will be summarised below. The reader is directed to the Section 6.3.2, which details the loading protocol for S01 and S02, for further details.

Specimens S03 to S05 had the same configuration as the cast in-situ building core specimen (i.e. S02) and were similarly tested with an axial load of -1200 kN, applied in the negative z-axis of the machine in a force-controlled mode. This axial load was applied at the beginning of the test and held for the whole duration. The test was terminated when the specimen could no longer support the initial axial load (i.e. complete structural collapse occurred) or when significant lateral strength degradation occurred. After the axial load was initially applied, the specimen was subjected to incrementally increasing in-plane lateral displacements in the x-axis.

Each specimen was subjected to two positive and negative cycles for each displacement increment (as shown in Figure 7.17). An in-plane moment about the y-axis was applied for the duration of the test equal to 5.2 times the lateral strength of the specimen (i.e. $M_y = 5.2 F_x$). This moment allowed the one-storey test specimen to be tested with a shear-span ratio of 6.5, which meant the specimen mimicked response of the ground storey component of a taller four-storey wall, as discussed above and further illustrated in Figure 6.3. The remaining out-of-plane DOFs were commanded to zero displacement and zero rotation for the duration of the test. A summary of the six DOF loading protocols is presented in Table 7.6. The x, y and z-axis setup that was adopted in the MAST System for this testing is shown overlayed on the specimens in Figures 7.6 and 7.7.

### Table 7.6: The MAST System loading protocol summary – test specimens S03 to S05.

<table>
<thead>
<tr>
<th>Degree of freedom</th>
<th>Mode</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_x$ – x-axis translation</td>
<td>Displacement</td>
<td>Figure 7.17</td>
</tr>
<tr>
<td>$T_y$ – y-axis translation</td>
<td>Displacement</td>
<td>Zero movement</td>
</tr>
<tr>
<td>$T_z$ – z-axis translation</td>
<td>Force</td>
<td>-1200 kN</td>
</tr>
<tr>
<td>$R_x$ – x-axis rotation</td>
<td>Displacement</td>
<td>Zero rotation</td>
</tr>
<tr>
<td>$R_y$ – y-axis rotation</td>
<td>Force</td>
<td>$M_y = k F_x$ *</td>
</tr>
<tr>
<td>$R_z$ – z-axis rotation</td>
<td>Displacement</td>
<td>Zero rotation</td>
</tr>
</tbody>
</table>

\* The specimens were tested with a shear-span ratio of 6.5, which resulted in a $k$ factor of 5.2 (as discussed in Section 6.1 and illustrated in Figure 6.3).

A single axial load ratio for each specimen cannot precisely be calculated, similar to what was presented in Chapter 6 for specimens S01 and S02, due to the varying concrete strength of the individual panels comprising each precast core specimens (i.e. Table 7.1). The axial load ratio is therefore presented as a range and a weighted average for specimens S03 to S05. The former was calculated by using the highest and lowest concrete strength for the respective panels used to construct specimens S03 to S05. Whereas the latter was calculated by determining a weighted average concrete strength for each specimen, which was equal to the average of the concrete strength of the four panels, adjusted by the cross-sectional area of each respective panel. The axial load ratio ranges were 5.2–6.1%, 5.5–5.7% and 5.0–6.3% for specimens S03, S04 and S05 respectively. While the weighted average axial load ratios were 5.6%, 5.7% and 5.3% for specimens S03, S04 and S05 respectively.
Figure 7.17: Cyclic x direction displacement increments – test specimens S03 to S05.

An important aspect of the test setup to note is that the top boundary element of each specimen (refer Figure 7.6) will provide additional restraint between adjacent panels, which will limit the amount of vertical displacement between the panels. Essentially this will create additional ‘coupling’ behaviour between adjacent panels and allow better composite action to be developed
in the core than would otherwise be developed if there were just welded stitch plates alone connecting the panels. This behaviour however, is not necessarily unrealistic or unrepresentative of the behaviour that would be seen in a real building. Many RC wall buildings in Australia that utilise precast walls have floor plates that are constructed using cast in-situ flat slabs (refer Figure 3.2). The flat slabs are usually either poured ‘over the top’ of the wall with vertical dowels running through the slab to the panel above or ‘hard up against’ the side of the panels with a 30 mm (give or take) shear key recessed into the face of the panel, with additional drag bars from the panel that are cast into the slab. The floor slab in this scenario will provide some additional coupling behaviour between adjacent panels, similar to the top boundary element in these test specimens.

7.2.4 Test Results

This section will outline the test results for specimens S03, S04 and S05. The general lateral displacement performance of all three specimens will be discussed initially, followed by an in-depth analysis of the results where the focus is test specimen S05. The latter will discuss the following in six separate sub-sections: crack propagation and curvature distribution; stitch plate performance; equivalent single DOF response and effective stiffness; displacement ductility and overstrength; deformation components, i.e. flexure, shear and sliding shear; and overall composite behaviour. The force-displacement behaviour, at the top of the specimens, are presented in Figures 7.19, 7.21 and 7.23 for test specimens S03, S04 and S05 respectively. Similarly, the moment-rotation and lateral displacement-axial displacement behaviour, at the top of the specimens, are presented in Figures 7.20, 7.22 and 7.24 for test specimens S03, S04 and S05 respectively. These plots only show the specimen response up until significant strength degradation occurred, i.e. the lateral strength dropped below 50% of the maximum lateral strength, for clarity. The specimens were subjected to larger in-plane lateral drifts than what is shown in Figures 7.19, 7.21 and 7.23 to better understand when complete structural failure occurs, i.e. the element cannot hold the initial axial load.

Varying levels of performance was observed in the three test specimens. Lateral load failure, which was defined to be when the lateral strength degraded below 80% of the maximum, occurred after lateral drift cycles of +2.4%/-2.1%, +2.1%/-1.9% and +1.1%/-1.2% for specimens S03, S04 and S05 respectively were exceeded. Overall failure, which was defined to be when the lateral strength degraded to zero, occurred after lateral drift cycles +4.6%/-4.8%, +8.2%/-8.0% and +3.5%/-3.4% for test specimens S03, S04 and S05 respectively. It can be noted from these figures that the actual respective positive and negative lateral drift value for each displacement increment (i.e. loading cycle) was different. This was due to the lateral displacement being commanded in the MAST System using the absolute x-axis displacement of the crosshead (i.e. a ‘machine’ displacement), as opposed to controlling the test using relative displacements obtained from independently mounted instrumentation. This is discussed further in Section 6.3.2.

Axial load failure of the specimens, i.e. complete structural collapse, did not occur in any of the specimens. Specimens S03, S04 and S05 were subjected to +7.1%/-6.5%, +8.2%/-8.0% and +4.9%/-4.8% respectively and were all able to withstand the initial axial of -1200 kN. The test was terminated after each of these respective drift cycles. This further shows that RC elements with low axial load ratios can withstand significant in-plane lateral drifts before complete structural collapse, which was discussed previously in Chapter 3 and highlighted in a recent study on RC columns by Wilson et al. [W9].

Photos showing the condition specimens S03 to S05 at the peak positive and negative strengths are presented in Figure 7.18.
All three specimens had force-displacement hysteresis curves that were not overly ‘stable’ and were subjected to various premature failure mechanism, which could usually be avoided through good reinforcement detailing. Specimens S03 and S04 (due to the construction problems discussion earlier), had a weak section at the base of the wall due to the percentage of dowel bar reinforcement being relatively much smaller than the vertical reinforcement in the panels. This meant that all the inelastic plastic behaviour was concentrated at the base of wall in one location – similar to what is seen in lightly reinforcement cast in-situ walls (e.g. [L9, L10]) – allowing very large tensile strains to be developed in the dowel bars. This resulted in the first dowel bar fracturing in specimen S03 at approximately -2.4% lateral drift (Figure 7.19). If the area of dowel reinforcement was higher than the vertical reinforcement in the panel, the plastic behaviour could have potentially been distributed over many cracks allowing for increased displacement capacity.

Figure 7.18: Photos of specimens S03 to S05 condition at peak strength.
Figure 7.19: Specimen S03 force-displacement behaviour at the top of the specimen.

Figure 7.20: Left – specimen S03 moment-rotation behaviour at the top of the specimen. Right – axial displacement-lateral displacement behaviour at the top of the specimen.

It should be noted though, that similar to what was observed in the cast in-situ wall testing in Chapter 6, the grout tube connection will create a localised region of overstrength. This means the area of dowel reinforcement should be equal to the area of panel reinforcement multiplied by the ratio of the moment at the base of the wall divided by the moment at the top of the grout tube to ensure a plastic hinge with distributed plasticity can actually be developed. The positive and negative strength of S03 was unsymmetrical (as seen in 7.19 and 7.20) because a dowel bar at one end of the web panel failed prematurely (as shown in Figure 7.8 and discussed in Section 7.2.1).
Lateral load failure of specimen S04 occurred at -3.0% lateral drift when two dowel bars at the base of the wall fractured and the strength dropped to approximately 65% of the maximum (Figure 7.21). Lateral load failure occurred at a higher drift in the positive direction and was due to a splitting failure of the compression flange panel. This was due to stress concentrations caused by poor grouting between the base of the panel and the top of the foundation block. Nominal cross ties at the base of the panel and improved grouting would prevent this type of failure mode.
Specimen S05 failed via fracturing of the vertical reinforcement at the first crack above the grout tube connection on the first positive load cycle that exceeded 1.1% lateral drift. Bar fracture at such a small amount of drift was caused by the panels being predominantly detailed using low ductile reinforcement (i.e. D500L mesh), which is common practice for precast RC wall panels in Australia. The majority of the plastic strains were concentrated in one crack, despite multiple cracks initially developing, due to the low strain hardening ratio of the D500L mesh. The same behaviour was observed in test specimen P17 and is discussed further in Section 5.3.2.
The lateral strength of S05 dropped to about 70% of the maximum after fracturing of the mesh in the flange panels and was maintained at this level because of the contribution from the two normal ductility bars at each end of the flange panels (i.e. the 2-N12 perimeters bar) and the mesh in the web panel of the specimen (as only the mesh in the flange panel fractured). On the second reversed load cycle to -1.2%, the N12 perimeter bars began to buckle (Figure 7.25(a)) and then on the next loading increment to -1.6%, the perimeter bars completely buckled and removed a significant portion of the wall surrounding these bars (Figure 7.25(b)). This suggests that if the wall did not have a premature failure due to fracturing of the low ductility mesh, the failure mechanism would have been local buckling of the vertical reinforcement. This provides large scale system level validation of the local bar buckling behaviour illustrated in the boundary element prim testing documented in Chapter 5. It should be noted that if the panels in S05 were detailed using solely normal ductility reinforcement (i.e. D500N bars), bar buckling would have likely occurred at a larger in-plane lateral drift. This is because the D500N bars would have allowed a better distribution of the inelastic plastic strains across multiple cracks, as D500N bars have a much greater strain hardening ratio than that of D500L mesh, meaning a larger in-plane displacement would have been required to allow the equivalent localised tensile strain to be developed that trigger the buckling behaviour of the perimeter bars.

![Figure 7.25: Bar buckling in test specimen S05.](image)

Test specimens S03, S04 and S05 all experienced minimal strength degradation between the first and second loading cycle for each respective displacement increment. This is illustrated in Figure 7.26, which shows the backbone force-displacement curve for each loading cycle. Significant strength degradation between loading cycles typically started to occur after lateral load failure of the specimen had occurred (i.e. the lateral strength had degraded below 80% of the maximum lateral strength). Otherwise, the strength degradation between loading cycles was typically less than about 15%. The peak displacement, drift and force values to construct the backbone curves in Figure 7.26 are summarised and tabulated in Appendix D for the convenience of the reader.

Photos showing the damage progression of test specimen S05 throughout the duration of the test are presented in Appendix D.
7.2.4.1 Crack Progression and Curvature Distribution

Test specimens S03, S04 and S05 were all detailed using moderate to high percentages of vertical reinforcement, meaning well distributed closely spaced cracking was expected (as discussed in Section 3.5.4). Crack map summaries for each test specimen are presented in Figure 7.28, which shows closely spaced evenly distributed cracking in test specimens S03 and S05 and sparse unevenly spaced cracking in test specimen S04. The area of dowel reinforcement at the base of the wall in test specimens S03 and S04 was significantly less than the vertical reinforcement in each respective panel due to the construction errors discussed previously (refer Section 7.2.1). This resulted in a ‘weak region’ at the base of wall, which formed into a single crack hinge, allowing a rocking mechanism to develop. This meant all the inelastic plastic response of test specimen S03 and S04 was concentrated in this single crack at the base of wall, as illustrated in the curvature distributions for each respective specimen shown in Figure 7.27. The curvature distributions in Figure 7.27 were calculated using the LVDT stacks at each end of the wall and as such, only show the curvature distributions for half the height of the specimens. The negative curvature for test specimen S04 in load cycle 103 shown in Figure 7.27 is not representative of the actual response due to localised cracking that occurred around the LVDT on that load cycle.

Figure 7.27: Test specimen S03 (left) and S04 (right) curvature distributions.
Figure 7.28: Test specimens S03 to S05 crack map summary.
The sparse unevenly spaced cracking shown in Figure 7.28(b) for test specimen S04 is believed to be misleading. Due to the very high percentage of vertical reinforcement that was a result of the construction errors discussed previously, it is believed that the panels (particularly the flange panels of the building core) developed closely spaced hairline cracks, which were too fine to be seen through the white paint that the specimens were coated in prior to testing. The progression of cracking throughout the duration of the test was documented and is presented in Appendix D.

The dowel connection at the base of Test specimen S05 resulted in the two-crack plastic hinge behaviour that was observed and discussed in Chapter 6. This is illustrated in the positive and negative curvature distributions shown in Figures 7.29 and 7.30.

**Figure 7.29:** Test specimen S05, positive loading cycles. Left – curvature distribution. Middle – tension strain distribution. Right – compression strain distribution.

**Figure 7.30:** Test specimen S05, negative loading cycles. Left – curvature distribution. Middle – tension strain distribution. Right – compression strain distribution.
Interestingly, the inelastic plastic strains in the positive loading directions were predominantly concentrated in two cracks; the first at the base of the wall and second at the top of the dowel connection. Whereas in the negative loading direction, the inelastic plastic strains were concentrated between a single crack at the base of the wall and over multiple cracks directly above the dowel connection. The same response was observed in the cast in-situ building core specimen, i.e. test specimen S02 (refer Figures 6.23 and 6.24). The curvature and strain distributions in Figures 7.29 and 7.30 were determined using the contactless photogrammetry system. The strain distributions represent the extreme axis tensile and compressive strain in the tension and compression flange panel of the building core respectively. The curvature distributions were calculated directly from the strain distributions.

### 7.2.4.2 Stitch Plate Performance

Test specimens S03 and S04 were constructed using ‘inside fixed’ welded stitch plates, whereas test specimen S05 was constructed using ‘outside fixed’ welded stitch plates (refer Figure 7.9). In addition to the construction errors discussed in Section 3.5.4 for test specimens S03 and S04, the stitch plates were also constructed slightly differently to what was specified in the original construction drawings (as shown in Appendix A). Firstly, the cast-in plates on the web panel were positioned horizontally instead of vertically, which meant one of the shear studs was on the ‘outside’ of the further most vertical reinforcing bar. Secondly, the ‘U’ bar that was specified to wrap around the shear studs (refer construction drawings in Appendix A) was omitted. The actual configuration of the stitch plate connection (i.e. the stud locations relative to the vertical reinforcement) was determined by locally demolishing the concrete around the connection after the testing was completed. This different cast-in plate configuration meant that the plate was able to rotate and split the vertical edge of the panel. This mechanism was observed in test specimen S04 between lateral drifts +1.4/-1.2% and +2.1/1.9%, as shown in Figure 7.31. The vertical splitting of the web panel in S04 can also be seen in the crack map summary in Figure 7.28(b) and the crack progression maps in Appendix D.

![web panel elevation](image)

(a) actual cast-in plate configuration

(b) load cycle 95, +2.1% lateral drift

**Figure 7.31:** Stitch plate failure in test specimen S04.
It was originally intended to calculate the force transferred through each stitch plate in test specimen S05 using the contactless photogrammetry system and a finite element (FE) model of the connection. An elastic FE model of the connection was constructed using Strand7, a general-purpose finite element analysis (FEA) system for structural analysis. Strand7 is a widely used structural analysis program in Australia, which was first developed in the late nineties. Eighteen prescribed node displacements were applied to the connection that corresponded to the photogrammetry markers placed on the respective stitch plate (refer Figure 7.13). The force transferred across the stitch plate could then be calculated by assessing the stresses developed in the plate by the movement from the prescribed node displacements. The FE model of the stitch plate connection is shown in Figure 7.32.

Figure 7.32: Finite element model of welded stitch plate connection.

This procedure was initially calibrated using the component level connection test of the WSP connection, which is presented later in Section 7.3. It was found that the photogrammetry system did not have sufficient resolution to accurately capture the relative movement from one side of the stitch plate to the other. While the overall connection underwent significant movement during the test in the form of rigid body rotation, the relative movement of the stitch plate was in the vicinity of 0.01 mm, which is the approximate shear displacement of a 16x150 mm steel plate spanning a length of 20 mm (i.e. the panel gap) subject to a load of 90 kN (i.e. approximately the maximum capacity of the connection as determined later in Section 7.3). The shear displacement can be taken as $\Delta_s = \frac{FL}{GA}$ and therefore: $90 \times 10^3 \times 20 /(80,000 \times 160 \times 50) = 0.01$ mm. This meant the FE model was not capable of predicting the forces transferred by each stitch plate connection.

An alternative procedure was therefore needed to determine the forces transferred across each stitch plate. A backbone curve of the overall force-displacement behaviour for the WSP connection was determined using the results of the component level test presented later in Section 7.3 (i.e. Figure 7.55, where the force on the backbone curve corresponds to the amount of vertical shear force transferred across the WSP connection and the displacement corresponds to the differential vertical movement of the adjacent precast panels). The force transferred through each stitch plate connection was calculated using this backbone curve and the differential movement between the panels that was determined by the photogrammetry markers above and below each stitch plate connection, as illustrated visually in Figure 7.33. The resolution of the photogrammetry system was sufficient for this approach as the differential panel movement varied from 0.2 to 4.0 mm.
The connection force was calculated for all eight WSP connections in specimen S05 for loading cycles 15, 17, 35, 37, 55, 57, 75 and 77 (the lateral displacement and drifts these load cycles correspond to are shown in Figures 7.17(c) and 7.23). The connection forces for the four front WSP connections are shown in Figure 7.33, while the connection force and displacement (i.e. differential panel movement) of all eight WSP connections are tabulated in Appendix D.

Figure 7.33 shows that the maximum capacity of the rear face WSP connections was not exceeded during load cycle 75 and 77, which corresponds to a lateral drift of +1.1% and -1.2% respectively and the maximum capacity of the specimen being reached (as shown Figure 7.23). Although it should be noted that the force in the bottom WSP connection on the tension face of the specimen (i.e. stitch plate #3 during load cycle 77 and stitch plate #4 during load cycle 75) was only marginally less than the maximum 88 kN capacity of the connection that was experimentally determined in Section 7.3. The top two rear connections had smaller connection forces than the top two front connections and similarly, the bottom two rear connections also had smaller connection forces than the bottom two front connections (refer Appendix D).
7.2.4.3 Equivalent Single Degree-of-Freedom Response and Effective Stiffness

Test specimens S03, S04 and S05 represent the ground story component of a taller four-storey wall, as further discussed in Chapter 6 and illustrated in Figure 6.3. This test setup has become a widely used testing procedure for RC walls in recent years as it allows for walls to be tested with high shear-span ratios that would otherwise be logistically very difficult or impractically large test specimens. However, it means the force-displacement response of these test specimens (e.g. Figures 7.19, 7.21 or 7.23) only represent the ground floor response (i.e. Figure 7.34(b)) of the equivalent four-storey wall (i.e. Figure 7.34(a)). However, when assessing the performance of the wall, e.g. effective stiffness, displacement ductility or unpacking the different components of deformations, the effective response (i.e. equivalent single DOF system response), should be used (i.e. Figure 7.34(c)), otherwise misleading results or outcomes could be reported. This was shown and further discussed in Sections 6.4.3 and 6.4.4.

![Figure 7.34: Test specimen response versus equivalent 1-DOF system.](image)

The equivalent single DOF response for test specimen S05 was calculated using the process outlined in Section 6.4.2 (i.e. Equation 6.4), where the flexure displacement is calculated by double integrating a theoretical average curvature distribution for the top two thirds of the wall using an empirical curvature relationship (i.e. Figure 7.35) and the shear displacement is calculated using an empirical shear modulus relationship (i.e. Figure 7.36). The empirical relationships were calculated using the elastic or pseudo-elastic response regions of the specimen, i.e. regions where significant plastic deformation occurred (e.g. the base of the wall or the top of the dowel connection) were excluded and only the pseudo-elastic response cycles were used (i.e. 15, 17, 35, 37, 55, 57, 75 and 77).

Specimen S05 could essentially be split into four different regions: the first is the dowel region with an opening at the base of the wall; the second is a general region (i.e. between the dowel regions) with an opening; the third is a general region also, however without an opening; and the fourth is the dowel region without an opening at the top of the wall. The four regions can be seen visually in Figure 7.37. It was observed while developing the empirical relationships that the opening had little effect on the average moment-curvature response of the cross-section and similarly, the dowel region had little effect on the average shear modulus relationship. Therefore, only two empirical models were required for the curvature and shear modulus relationships, resulting in curvature-moment empirical relationships for dowel regions and general regions (i.e. Figures 7.35(a) and 7.35(b) respectively) and shear modulus-moment relationships for regions with and without openings (i.e. Figures 7.36(a) and 7.36(b) respectively).
Figure 7.35: Empirical curvature-moment relationship for test specimen S05.

Figure 7.36: Empirical shear modulus-moment relationship for test specimen S05.
The theoretical top two-thirds segment of the wall was split into 40 segments, each therefore being 130 mm high. The average moment acting at each of the 40 segments was calculated for each time step of the test using the x-axis force corresponding to that time step. The average curvature for each segment was then calculated using the corresponding average moment for that segment and the relationships presented in Figure 7.35. A different curvature relationship was used for the general regions and dowel regions of the wall, i.e. Figures 7.35(a) and 7.35(b) respectively, as discussed previously. The general regions and dowels regions for the theoretical top two-thirds segment of the wall are shown in Figure 7.37. This allowed an averaged curvature distribution to be determined for each time step that could then be double integrated to calculate the flexural deformation component of the theoretical top two-thirds segment of the wall.

Similarly, the shear displacement was calculated for each segment using the corresponding shear modulus calculated for that segment. A different shear modulus relationship was used for the regions with and without openings, i.e. Figures 7.36(a) and 7.36(b) respectively, as discussed previously. The different regions with and without openings are shown in Figure 7.37. The shear displacement component of the theoretical top two-thirds segment of the wall was then calculated by summing the shear displacement of the 40 individual segments. The lateral displacement of the equivalent single DOF wall is then equal to the lateral displacement of the test specimen plus the flexural and shear components calculated for the theoretical top two-thirds segment of the wall. The force-displacement response for the equivalent single DOF system of specimen S05 is presented in Figure 7.38.

![Figure 7.37: Equivalent single DOF wall, i.e. effective response of four-storey wall.](image-url)
The force-displacement response of the equivalent single DOF wall (i.e. Figure 7.37) for specimen S05 is presented in Figure 7.38. Figure 7.38 also presents a bilinear response of S05, which has been constructed as per procedure set out in Section 6.4.4. The ‘up-branch’ of the bilinear relationship is taken as a line that runs from the origin through the point that corresponds to the first yield moment being reach. The definition of the first yield moment is presented in Section 6.4.4. The dowel reinforcement and vertical reinforcement in S05 is different, which means the yield moment and maximum moment capacity at the base of the wall and the top of the dowel connection will not be the same. This means the yield force capacity of S05 is equal to the lateral force corresponding to the yield moment being reach at the base of the wall or the top of the dowel connection, whichever occurs first.

The yield moment and maximum moment capacity at the base of the wall and top of the dowel connection for test specimen S05 was calculated using the fibre-element analysis method described in Section 4.3.2 and its material properties presented in Tables 7.1 and 7.5. It should be noted that the fibre-element analysis procedure assumes full composite action between the panels, not allowing for the flexibility of the WSP connection to be considered. The yield moment and maximum moment at the base of the wall was 2,435 and 3,025 kNm respectively and similarly, at the top of the connection it was 2,010 and 2,700 kNm respectively. The corresponding force to develop the yield moment at the base of the wall and the top of the dowel connection is then 287 and 312 kN respectively, which means the yield force capacity of S05 is 287 kN and is governed by the yield moment capacity being reached at the top of the dowel connection. The bilinear response curve data for S05 is summarised in Table 7.7.

![Figure 7.38: Equivalent single DOF force-displacement response of test specimen S05.](image)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$M_y$ (kNm)</th>
<th>$F_y$ (kN)</th>
<th>$\Delta'_y$ (mm)</th>
<th>$F_{max}$ (kN)</th>
<th>$\Delta_y$ (mm)</th>
<th>$\Delta_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S05</td>
<td>2010*</td>
<td>287</td>
<td>68.9</td>
<td>320</td>
<td>76.7</td>
<td>96.9</td>
</tr>
</tbody>
</table>

* The theoretical yield moment was governed by yielding of the extreme tensile reinforcement of wall directly above the dowel connection, i.e. $F_y = 2010/(7.8 - 0.8) = 287$ kN.
The effective stiffness of S05 is then equal to the slope up-branch of the bilinear response shown Figure 7.38 and the effective moment of inertia (i.e. $I_{eff}$) can be calculated using Equation 6.6 and the modulus of elasticity for the concrete. The average modulus of elasticity of the concrete used to construct S05 was calculated to be 33,100 MPa, which was determined using the equation from AS 3600 and the concrete compressive strengths presented in Table 7.1. The ratio of the effective to gross moment of inertia was then calculated to be 0.17 (refer Table 7.8). This is 25% lower than the equivalent ratio for the cast in-situ building core specimen (i.e. S02). Meaning the jointed precast building core specimen constructed using WSP connections is 25% more flexible than the cast in-situ building core specimen.

**Table 7.8:** Effective moment of inertia.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$I_{eff}$ (m$^4$)</th>
<th>$I_g$ (m$^4$)</th>
<th>$I_{eff}/I_g$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>0.0089</td>
<td>0.0288</td>
<td>0.32</td>
</tr>
<tr>
<td>S02</td>
<td>0.0259</td>
<td>0.1064</td>
<td>0.24</td>
</tr>
<tr>
<td>S05</td>
<td>0.0200</td>
<td>0.1142</td>
<td>0.17</td>
</tr>
</tbody>
</table>

### 7.2.4.4 Displacement Ductility and Overstrength

The overstrength and displacement ductility of test specimen S05 was calculated to be 1.1 and 1.3 respectively (Figure 7.38 and Table 7.9). These values are somewhat smaller than the ‘default’ values of 1.3 and 2.0, respectively, that AS 1170.4 allows for RC structures. The resulting force reduction factor for S05 was 1.4, which is significantly less than the value of 2.6 that AS 1170.4 allows designers to currently adopt. The small displacement ductility is attributed to the low ductility reinforcement, while it is believed the smaller overstrength factor was due to the walls inability to develop ‘full’ composite action, meaning the theoretical maximum moment capacity of the section could not be developed, hence reducing the amount of overstrength the wall could develop.

It should be noted that comparing the overstrength (i.e. $\Omega$), ductility (i.e. $\mu$) and force reduction (i.e. $R_f$) factors from AS 1170.4 to the results of test specimens S01 and S02 (refer Chapter 6) or S05 can be somewhat misleading since these factors for these specimens are calculated using their actual yield displacements. Whereas when force-based analysis is performed using AS 1170.4 the yield displacement of the building or structure is essentially controlled by the empirical equation used for calculating the first mode natural period (i.e. Equation 2.7, refer Section 2.4.2), which is a very conservative equation. Therefore, AS 1170.4 underestimates the yield displacement (as a result of the conservative natural period calculation), which means higher force reduction factors would be required to factor up the smaller yield displacement to achieve the equivalent ultimate displacement.

**Table 7.9:** Test specimen S05 force-based analysis coefficients.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\Omega$</th>
<th>$\mu$</th>
<th>$R_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S05</td>
<td>1.1</td>
<td>1.3</td>
<td>1.4</td>
</tr>
</tbody>
</table>
7.2.4.5 Deformation Components

The different components of deformation for test specimen S05 was determined using the contactless photogrammetry system. The flexure deformation was calculated by double-integrating the curvature distributions (i.e. Figures 7.29 and 7.30) and the sliding shear deformations was determined directly from string potentiometers at the base of the wall (i.e. Figure 7.16). The shear deformation was then taken as the difference between the overall displacement and the combined flexural and sliding shear deformations, as shown in Figure 6.33(b). The percentage of each deformation component (i.e. flexure, shear and sliding shear deformation) with respect to the amount of in-plane lateral drift for test specimen S05 is presented in Figure 7.39. Similarly, the percentage of each deformation component for the response of the equivalent single DOF wall developed above (i.e. Figures 7.37 and 7.38) is presented in Figure 7.40. It should be noted the deformation components calculated are for the entire system (i.e. jointed building core) and not for individual panels.

The percentage contribution of each component of deformation varies significantly between Figures 7.39 and 7.40, where the percentage of flexural deformation increases from around 50% when just the test specimen response is considered, to about 70% when the equivalent single DOF response is considered. Test specimen S05 (i.e. the jointed building core specimen) had a larger percentage component of shear deformation than test specimen S02 (i.e. the cast in-situ building core specimen), which had a higher percentage of flexural deformation of about 90% higher. Test specimen S05 had marginal sliding shear deformations. Additional displacement profiles for test specimen S05 are provided in Appendix D.

Figure 7.39: Deformation components – test specimen S05 – test specimen response.

Figure 7.40: Deformation components – test specimen S05 – equivalent single DOF response.
7.2.4.6 Overall Composite Behaviour

It was shown in the previous sections that the jointed precast building core specimen (i.e. S05) was significantly more flexible than the cast in-situ building core specimen (i.e. S02), implying that the WSP connections were not stiff enough to allow full composite action to develop. This is confirmed by Figure 7.42, which shows curvature profiles of specimen S05 for two positive and negative loading cycles. Curvature profiles are presented for a section of wall above the bottom stitch plates and at the base of the specimen, as shown in Figure 7.41, and were calculated using the photogrammetry system. It is shown in Figure 7.42 that the maximum compressive and tensile strains in the web panel of S05 are greater than the compressive and tensile strains in the corresponding compressive and tensile flange panels of the building core. This can also be seen visually in Figure 7.41, which shows the base of the web panel at load cycle 77 (i.e. -1.2% lateral drift), where the crack width at the tension end of the web panel is wider than the crack at the base of the tension flange. Additionally, spalling of the grout at the compression end of the web panel has occurred, while the base of the compression flange is fully intact without any spalling. Damage progression photos of the bottom web panel stitch plates (matching Figure 7.41) for each loading cycle are provided in Appendix D.

In Section 7.2.4.2 it was shown that stitch plate #3 and #4 (refer Figure 7.33) were close-to or on the verge of failing during load cycle 77 and 75 respectively. Despite this, it is clear that the WSP connections were not stiff enough to allow full composite action to developed, as Figures 7.42(c) and 7.42(d) show that it was not able to be fully developed during load cycles 55 or 57 (i.e. ±0.6% lateral drift), during which the WSP were exhibiting an elastic response and the force in each connection did not exceed more than about 70% of its maximum capacity (refer Figure 7.33).

![Damage progression photos of the bottom web panel stitch plates](image)

*Figure 7.41: Test specimen S05, web panel elevation, load cycle 77, -1.2% lateral drift.*
Figure 7.42: Curvature profiles at the base of test specimen S05.
In Section 7.2.4.3 it was determined that the maximum moment capacity at the base of the wall and the top of the dowel connection was 3,025 kNm and 2,700 kNm respectively. This corresponds to lateral forces of 388 and 386 kN respectively (i.e. 3,025/7.8 = 388 and 2,700/7.0 = 386 kN respectively), inferring that the maximum lateral strength of S05 should have been approximately 386 kN. However, the maximum strength of S05, prior to the vertical reinforcement fracturing, was 302 and 314 kN in the positive and negative loading directions respectively. Meaning only 80% of the theoretical maximum moment capacity of the wall was able to be developed. This further indicates full composite action could not be developed.

### 7.2.5 Conclusions and Recommendations

The following conclusions and recommendations have been made following the system level precast building core experimental testing program:

1. Poor grouting at the base of the panels can result in stress concentrations and premature compression failures due to vertical tensile splitting of the compression flange panel. This would likely be mitigated by providing nominal cross ties at the top and bottom of the panel, which would mean the panel is then not relying on the concrete’s tensile strength to prevent these potential tensile splitting failures. It is therefore being recommended to provide a cross tie at each grout tube connection that has a cross-sectional area at least one quarter that of the respective dowel bar.

2. The load is not evenly distributed amongst the stitch plates and is concentrated towards the connections at the base of the wall on the web panel. Further analytical work is recommended to better understand how the load is distributed.

3. The WSP connections were not stiff enough to allow full composite action to be developed. This resulted in the effective moment of inertia of test specimen S05 being 25% lower than the cast in-situ building core specimen tested in Chapter 6 (i.e. S02). Further, it meant that only 80% of the theoretical maximum moment capacity of the wall could be developed before flexure failure occurred via fracturing of the vertical reinforcement.

4. Low ductility reinforcement allows only marginal ductility to be developed before fracturing of the reinforcement occurs and results in a sudden reduction in lateral strength. However, the lateral strength does not drop to zero, as the web panel and the nominal perimeter bars, which are normal ductility reinforcement, still provide some level of resistance. This provides further evidence to support the ductility classifications discussed and recommended in Chapter 3.

5. One large crack develops with all the inelastic plastic tensile strains concentrated in this single crack if the dowel bar area is less than the area of vertical reinforcement in the panels. This allowed a rocking mechanism to develop, which then allowed for very large in-plane lateral drifts without axial load failure of the wall occurring. However, this mechanism results in a weak region at the base of the wall that would likely have little resistance to torsional actions since the section at this location has developed large plastic tensile strains from the in-plane response. Torsional actions are common design actions for building cores in multi-storey buildings and therefore it is being recommended to ensure the dowel bar area is greater than the area of vertical bars in all situations.

6. Despite the lateral strength decreasing significantly, often below 20% of the maximum response, the walls could withstand very large in-plane lateral drifts prior to axial load failure occurring (i.e. complete structural collapse). Test specimens S03, S04 and S05 were loaded to 6.5%, 8.0% and 4.9% respectively without axial load failure occurring, at which point the test was terminated.
7.3 Precast Connection Testing – Test Specimens J01 to J03

This section outlines test specimen details, instrumentation, test setup, loading protocol and results of the component level precast building core connection specimens.

7.3.1 Test Specimen Overview

The experimental component level precast building core connection testing program consisted of three specimens denoted J01, J02 and J03. Each specimen consisted of two rectangular panels that were joined together to form the corner segment of a jointed precast building core, as discussed previously and shown in Figure 7.4. The first specimen (i.e. J01) was the baseline specimen and had ‘welded stitch plate’ (WSP) connections joining the two panels together. This connection was mostly identical to the WSP used in the system level building specimen S05. The second and third specimens (i.e. J02 and J03 respectively) were the new prototype connections, which have been developed to construct precast cores without the requirement for site welding. The connection details and 3D perspective views of all three specimens are presented in Figures 7.44 and 7.45 and Figure 7.46 respectively.

The first prototype specimen (i.e. J02) is being referred to as a ‘grouted panel pocket’ (GPP) connection. The second prototype specimen (i.e. J03) is being referred to as a ‘post tensioned corbel’ (PTC) connection. The GPP was developed with ease and speed of construction as the primary objectives, whereas the PTC was developed with strength and stiffness as the primary objectives. This resulted in the GPP connection being simpler and quicker to construct than the PTC connection, however it then had significantly less capacity and stiffness.

The GPP connection consisted of one panel with 80x300 mm voids along the vertical end of the panel and the second with M20 cast in ferrules along one vertical edge. When the two panels are erected adjacent to one another, the panel voids on the first panel line up with the cast in ferrules along the edge of the second panel. High tensile grade 8.8 M20 bolts are then inserted through the panel void and into the ferrules. The void is then boxed up and filled with high strength cementitious grout. Two different methods are being proposed for grouting the pocket and are illustrated in Figure 7.43. The first method involved fixing a rectangular piece of formply across the void on the outside of the panel and then fixing a ‘U’ shaped piece of formply between the 20 mm panel gap. Both pieces of formply were fixed using 6 mm concrete screws. The void was then filled with cementitious grout by pumping it through a pipe inserted between the panel gap and through the top of the ‘U’ section of formply into the void. The second method is to firstly seal the panel gap around the void using gap filler and then fix a rectangular piece of formply with an outlet hole across the back of the void. Grout is then pumped through the outlet hole in the formply. Gap filler is a widely used product in the precast industry and is commonly used to fill the gaps between starter bar shutters. The first method was used for constructing test specimen J02.

The PTC connection consisted of one panel that was the bottom section of the corbel and had a 26.5 mm nominal diameter Macalloy bar cast into it. The second panel was the top section of the corbel and had a 60 mm diameter corrugated grout tube cast into it. When erected, the Macalloy bar in the bottom section slotted through the corrugated grout tube in the top section. High early strength grout was used to dry pack the 20 mm horizontal gap between the top and bottom corbel sections. The day after the panel gap was dry packed with grout, the Macalloy bar was post tensioned. The grout tube was pumped full using a high strength cementitious grout immediately following the post tensioning of the Macalloy bar. The grout was pumped through a 15 mm diameter tube cast in at the bottom of the grout tube. A second 15 mm diameter tube was cast in at the top of the grout tube to give an indication of when the grout tube was full.
The panels were capacity designed to ensure failure of the connection could occur with only elastic response of the panels. This meant that when the panels were ‘pulled up’ in tension, the tensile stresses in the reinforcement remained below the characteristic yield stress of the bars, meaning no plastic tensile strains developed and similar, when the panels were ‘pushed down’ in compression, the compressive stress in the concrete remained below the maximum characteristic compressive strength of the concrete mix. This resulted in the panels having relatively ‘heavy’ percentages of vertical reinforcement, which equalled 2.3% and 1.8% for the 700 and 900 mm long panels respectively. All the horizontal reinforcement and ligatures in the panels were initially specified as N10, i.e. 10 mm nominal diameter grade D500N to AS/NZS 4671 reinforcing bars, however due to supply issues at the time all the N10 bars were substituted for N12 bars. All the panels were 150 mm thick, matching the system level test specimen denoted S05. The three specimens were delivered to the Smart Structures Laboratory at Swinburne as six individual panels, which were then assembled on site and connected together accordingly to create each respective test specimen. The construction drawings for each test specimen are provided in Appendix A.

Each respective precast panel of the three specimens were constructed using the same industry standard N40 precast mix that was used to construct the cast in-situ specimens (i.e. S01 and S02) and the precast specimens (i.e. S03 to S05). The N40 precast mix had a maximum aggregate size of 14 mm and a minimum characteristic 28-day compressive cylinder strength of 40 MPa. The six panels were poured by a local precast manufacturer and each respective panel for each test specimen were poured on separate days. This meant that each panel, for the same specimen, had a different concrete strength on test day. The concrete strengths on test day varied from 43 to 53 MPa for each panel and are summarised in Table 7.10.

Aitken Freemans Tecgrout HS, which has a 90 MPa characteristic compressive strength at 28 days, was adopted for the grouting of the GPP (i.e. J02) and the PTC (i.e. J03). Tecgrout HS has different water content requirements for trowellable and flowable mixtures. The flowable mix requires a higher water content and as such has a reduced maximum compressive stress compared to what would be achieved using a trowellable mix. A flowable mix was required as the grout needed to be pumped into the void and grout tube in specimens J02 and J03 respectively without air pockets forming. Grout samples were taken using 50 mm cube moulds. The grout strength on test day was 70 MPa.
Figure 7.44: Test specimens J01 and J02 connection details.
**Figure 7.45:** Test specimens J03 connection details.

**Figure 7.46:** Test specimens J01 to J03 perspective views.
Table 7.10: Concrete strength (precast wall component level tests).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Panel mark</th>
<th>Specified strength</th>
<th>Actual strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>J01</td>
<td>A</td>
<td>40 MPa</td>
<td>52.0 MPa</td>
</tr>
<tr>
<td>J01</td>
<td>B</td>
<td>40 MPa</td>
<td>49.0 MPa</td>
</tr>
<tr>
<td>J02</td>
<td>C</td>
<td>40 MPa</td>
<td>43.3 MPa</td>
</tr>
<tr>
<td>J02</td>
<td>D</td>
<td>40 MPa</td>
<td>52.9 MPa</td>
</tr>
<tr>
<td>J03</td>
<td>E</td>
<td>40 MPa</td>
<td>49.5 MPa</td>
</tr>
<tr>
<td>J03</td>
<td>F</td>
<td>40 MPa</td>
<td>44.6 MPa</td>
</tr>
</tbody>
</table>

The cast in plates used in the WSP connection (i.e. J01) were grade 250 to AS/NZS 3678, which has a minimum characteristic yield stress of 260 MPa for 12 mm thick elements. The stitch plates were grade 300 flat bar sections to AS/NZS 3679.1, which has a minimum characteristic yield stress of 300 MPa for 16 mm thick elements. Material samples of the structural steel plates and or flat bar sections for the WSP connections were not taken, however the steel plates and welds did not undergo any inelastic behaviour during the testing, maintaining an elastic response.

The bolts used in the GPP connection (i.e. J02) were grade 8.8 M20 all-threads with a class 8 nut at the end to ensure the threaded bar was completely developed and interlocked with the surrounding pocket of grout. The all-threads and associated nuts could equally be substituted for grade 8.8 M20 bolts, in which the bolt head would serve the same function as the nuts did. Grade 8.8 denotes a fastener with a nominal ultimate tensile stress of 800 MPa and a 'ratio of nominal yield strength and nominal tensile strength' of 0.8, as set out in AS 4291.1 [X47]. Specifically, a grade 8.8 M20 bolt or all-thread should have a minimum yield strength and tensile strength of 660 and 830 MPa respectively. A sample of the all-thread used in J02 was tested to determine its exact material properties, which were 830 and 922 MPa for the yield strength and tensile strength respectively (Figure 7.47(a)). M20 bolts and all-threads have a tensile stress area and core area of 245 and 225 mm² respectively. Meaning the yield and ultimate tensile capacity of the all-thread was 203 and 226 kN respectively and the ultimate shear capacity was 129 kN. The shear capacity is calculated by multiplying the core area of the threaded section by the ultimate tensile stress and a factor of 0.62 (AS 4100 [X7]). It is worth noting that despite both the yield and ultimate tensile strength being higher than the characteristic values, the actual yield stress to ultimate stress ratio was 0.9, which is significantly higher than the required 0.8.

The post tensioning bar in the PTC connection (i.e. J03) was a 26.5 mm nominal diameter Macalloy 1030 bar. Macalloy 1030 bars are 'fully threaded' commercially available high tensile post tensioning bars, which have been widely used in infrastructure and building projects around the world for many years. The 26.5 mm nominal diameter bar has a cross-sectional area of 572 mm² and an outside diameter of 30.4 mm, requiring a minimum 33 mm diameter hole through steelwork. The 25 to 40 mm nominal diameter bars have a nominal ultimate tensile strength, nominal 0.1% proof stress, minimum elongation and approximate modulus of elasticity of 1030 MPa, 835 MPa, 6% and 170,000 MPa respectively. The characteristic failing load and 0.1% proof load of a 26.5 mm bar are stated as 569 and 460 kN respectively by the manufacturer. These characteristic values are slightly smaller than what is determined by multiplying the bar area and nominal stress values given above. A sample of the 26.5 Macalloy 1030 bar was tested and 0.1% proof stress and ultimate stress was determined to be 821 and 1031 MPa respectively, or alternatively put, 470 and 590 kN respectively (Figure 7.47(b)).
During the assembly of the PTC connection a load washer was placed on the Macalloy bar between the load washer and the nut, as shown in Figure 7.45. The load washer was a 550 kN load cell and was being used to measure the amount of post tensioning force applied to the Macalloy bar prior to the bar being grouted in place. The post tensioning force applied prior to grouting was approximately 200 kN. The grout tube was grouted immediately following the post tensioning force being applied in the Macalloy bar. The intent with adopting a post tensioned corbel (as opposed to a corbel with a traditional grout tube style connection with a reinforcing bar as the dowel, i.e. Figure 7.2 or 7.11), is that the post tensioning would result in a stiffer connection and allow for more effective composite action to be established in a jointed precast core.

### 7.3.2 Instrumentation

A combination of physical instrumentation and the photogrammetry system discussed previously (i.e. the V-STARS N series by Geodetic Systems) was used to measure the response of the test specimens. Given the simplicity of the test (i.e. it was essentially just a 1-DOF test where the specimen is being ‘pulled’ and ‘pushed’ in axial tension and compression), only a minor amount of instrumentation was used, with a heavy reliance on the photogrammetry system to capture any complex or unaccepted behaviour. String potentiometers (SPOT) were used to measure the overall vertical displacement between the specimens in all three specimens. In test specimens J01 and J02 additional string potentiometers were fixed either side of the WSP and GPP connections in J01 and J02 respectively to capture localised deformation of each connection in each respective specimen. In test specimen J03 a combination of linear potentiometers (LPOT) and string potentiometers were used to measure the movement locally across the corbel interface and other areas where it was believed important cracks would develop. Figure 7.48 shows specimen elevations indicating where either horizontal or vertical displacements were measured between using physical instrumentation. A summary table of all the physical instrumentation is provided in Appendix D.

The loading protocol consisted of two positive and two negative cycles for loading increment. Photogrammetry measurements were taken for each specimen on the first positive and first negative loading cycle for each loading increment. The location of photogrammetry targets, where x-y-z movement was calculated and recorded as discrete moments throughout the test, for each specimen, is shown in Figure 7.49.
7.3.3 Test Setup and Loading Protocol

The precast connection specimens were tested in the MAST System, which was discussed briefly above in Section 7.2.3 and described in detail in Section 6.3.1. The test was essentially a 1-DOF experimental test setup, however the full 6-DOF capability of the MAST System allowed for state-of-the-art control of the five out-of-plane DOFs. Without this extra level of control afforded by the MAST System, the test setup for specimens of this nature would have been highly cumbersome. The ‘L’ shape specimens were essentially ‘pulled’ and ‘pushed’ in axial compression and tension in the z-axis (as shown in Figure 7.46) to impose a vertical shear force on the panel connections, similar to what would be observed in a system level response (as shown in Figure 7.4).
A series of structural steel plates and loading brackets were custom fabricated to connect the specimens to the MAST System and strong floor, while allowing the appropriate boundary element constraints to present during testing. The top of panels A, C and E (i.e. specimens J01, J02 and J03 respectively) were connected to the crosshead of the MAST System using two fully fixed connection brackets, one either side of the panel. The specimens were loaded vertically through these two brackets. The bottom of panels B, D and F (i.e. specimens J01, J02 and J03 respectively) were connected to structural steel support plates using two fully fixed connection brackets, one either side of the panel. The structural steel support plates were in turn fully fixed to the strong floor, providing the base restraint for the specimens. These two respective connections were the primary load and restraint points for the test setup.

The bottom of panels A, C and E and the tops of panels B, D and F then had two connection brackets, one either side of the panel, which had vertical slotted holes. These brackets allowed the panels to move vertically between them, however they restrained their respective panel from moving laterally out-of-plane in either plan direction or twisting about the z-axis, while the specimens were being loaded. This combination of fixed and vertically slotted connection brackets allowed the panel connections (i.e. WSP, GPP or PTC) to be subjected to purely vertical shear forces, similar to what would be observed in a system level response, without any out-of-plane forces developing due to the eccentric nature of the test setup. The test setup and loading brackets are shown and further described in Figure 7.50.

![Figure 7.50: The MAST System test setup for the precast connection testing.](image-url)
The five out-of-plane DOFs on the MAST System were commanded in a displacement-controlled mode to either zero movement or zero rotation for the duration of the test. The specimen was loaded through the z-axis in a displacement-controlled mode, however the amount of z-axis movement was controlled by either a string potentiometer attached to top of the ‘loading panel’ and the bottom of the ‘support panel’ (denoted L01-SPOT in Figure 7.48) or a ‘force trigger’. This meant the specimen could be loaded towards a pre-selected z-axis displacement and force value and the loading would stop when the first of the two values were reached.

The z-axis loading was applied in series that comprised two positive and two negative loading cycles. For test specimens J01 and J02 the positive and negative loading for each series was the same because both connections have a symmetrical response. J03 however, has an unsymmetrical response; under z-axis positive loading the corbel is pulled in tension and the Macalloy bar resists the load, while under z-axis negative loading the corbel is pushed in compression and a strut and tie mechanism in the corbel resists the load.

For the symmetrical response specimens (i.e. J01 and J02) the z-axis loading protocol consisted of two initial loading series where force triggers were used and the intent was that the response would be solely within the linear elastic range of the connection. Following these force cycles, it would switch to incrementally increasing loading series with displacement triggers. While testing the first specimen (i.e. J01), it was discovered that there was an error in the loading protocol program, as the specimen was loaded to the first displacement increment, skipping the fore increments. This error was rectified for the second specimen (i.e. J02) and the aforementioned discussed loading procedure was successfully adopted. The MAST System loading protocols for J01 and J02 are summarised in Table 7.11 and Figure 7.51.

For the unsymmetrical response specimens (i.e. J03) the z-axis loading protocol in the positive direction consisted of three incrementally increasing force cycles, followed by incrementally increasing displacement cycles. Whereas in the negative direction it consisted of seven incrementally increasing force cycles before softening of the corbel started to occur and the negative direction loading was switched to displacement triggered cycles. The MAST System loading protocol for J03 is summarised in Table 7.11 and Figure 7.51.

The incrementally increasing force and displacement values were generally selected with reference to the procedure used for the cast in-situ wall testing in Chapter 6, where it was proposed that subsequent displacement values would be between 5/4 and 3/2 times the current displacement value (refer Section 6.3.2). It is noted that this procedure was not strictly followed for the negative z-axis loading of test specimen J03, as the response of J03 was essentially governed by the positive z-axis loading.

### Table 7.11: The MAST System loading protocol summary – test specimens J01 to J03.

<table>
<thead>
<tr>
<th>Degree of freedom</th>
<th>Mode</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_x$ – x-axis translation</td>
<td>Displacement</td>
<td>Zero movement</td>
</tr>
<tr>
<td>$T_y$ – y-axis translation</td>
<td>Displacement</td>
<td>Zero movement</td>
</tr>
<tr>
<td>$T_z$ – z-axis translation</td>
<td>Displacement</td>
<td>Figure 7.51</td>
</tr>
<tr>
<td>$R_x$ – x-axis rotation</td>
<td>Displacement</td>
<td>Zero rotation</td>
</tr>
<tr>
<td>$R_y$ – y-axis rotation</td>
<td>Displacement</td>
<td>Zero rotation</td>
</tr>
<tr>
<td>$R_z$ – z-axis rotation</td>
<td>Displacement</td>
<td>Zero rotation</td>
</tr>
</tbody>
</table>
(a) test specimen J01

(b) test specimen J02

(c) test specimen J03

**Figure 7.51**: Cyclic z-axis loading protocol for test specimens J01 to J03.

The numbers shown above the vertical lines in the z-axis loading protocols in Figure 7.51 are loading cycle reference numbers.
7.3.4 Test Results

The test results of the three building core connection specimens will be discussed in five sub-sections as follows: Section 7.3.4.1 Connection Strength and General Behaviour; Section 7.3.4.2 Connection Stiffness; Section 7.3.4.3 Test Specimen J01 Failure Mechanism; Section 7.3.4.4 Test Specimen J02 Failure Mechanism; and Section 7.3.4.5 Test Specimen J03 Failure Mechanism.

7.3.4.1 Connection Strength and General Behaviour

Test specimen J01 (i.e. WSP connections) had a maximum strength of 176 and 177 kN in the positive and negative loading directions respectively. The WSP connections exhibited a reasonably ductile failure mode (Figure 7.52), allowing a fair amount of connection deformation and gradual decline in strength, due to yielding of the shear studs, before complete failure occurred (i.e. fracturing of the shear studs).

Test specimen J02 (i.e. GPP connections) had a maximum strength of 304 and 308 kN in the positive and negative loading directions respectively. The GPP connections exhibited a reasonably ductile failure mode (Figure 7.52), also allowing a fair amount of connection deformation after the maximum strength was exceeded. However, unlike J01, the strength declined more significantly between both the respective cycles in the same loading series increment and from loading series increment to increment. The failure of the connection was governed by a cyclic degradation of the grouted pocket and shear failure of the bolts.

![Figure 7.52: Force-displacement overall response of test specimens J01 (left) and J02 (right).](image)

Test specimen J03 (i.e. PTC connection) had a maximum strength of 619 and 1061 kN in the positive and negative loading directions respectively. The PTC connection exhibited a stable hysteresis response (Figure 7.53) and was able to develop significant levels of ductility in the positive direction (i.e. when the corbel was ‘pulled up’ in tension), while it had a very sudden failure in the negative direction (i.e. when the corbel was ‘pushed down’ in compression). The ductile response in the positive direction was due to the response being governed by the Macalloy bar being pulled in tension and exhibiting inelastic strain hardening. The brittle response in the negative direction was due to general degrading and failure of the grout pad at the corbel interface.
This was not believed to be due to the grouts compressive strength being exceeded but rather from cyclic degradation of the grout pad caused by yield penetration of the Macalloy bar being pulled in tension, which then induces transverse tensile stresses and vertical cracking in the grout. The maximum compressive stress in the grout at the corbel interface was conservatively calculated to be approximately 48 MPa (i.e. the Macalloy bar was assumed to take zero compression load, which would be false), when the compressive load of 1061 kN was being applied. This is significantly less than the compressive strength of the grout, which was determined to be 70 MPa (refer Section 7.3.1).

The maximum tensile force test specimen J03 could resist prior to fracturing of the Macalloy bar was 619 kN, which is 29 kN higher than the ultimate tensile strength of the Macalloy bar sample taken during manufacturing of the specimen. It was shown in Section 5.4 that tensile tests of reinforcing bar samples taken from the same sample of reinforcement typically have approximately the same ultimate tensile strength. Further, if the sample is subject to cyclic loading and allowed to buckle in compression, it typically failed at a lower ultimate tensile stress and strain value. While the latter (i.e. bar buckling) did not occur in J03 as the Macalloy bar was confined from buckling in the grout tube, the former suggests the maximum strength should have been around 590 kN. It is believed this additional capacity, or approximately 5% increase in strength, was due to friction rubbing of the panels against the slotted connection brackets at the base of panels A, C and E and the top of panels B, D and F (refer Figure 7.50).

![Figure 7.53: Force-displacement overall response of test specimen J03.](image)

The load distribution between the top and bottom WSP and GPP connection in test specimens J01 and J02 respectively was assessed by investigating the local movement of each connection using a string potentiometer connected above and below each connection on each respective panel (i.e. L02-SPOT and L03-SPOT in Figure 7.48). The movement of the top and bottom WSP in J01 was approximately equal for the duration of the test (Figure 7.54) and hence it was assumed that the force was evenly distributed across both connections. The movement of the top and bottom GPP in J02 however, was only approximately equal for the first two loading series increments (Figure 7.54). The maximum load though was essentially developed in the second loading series increment and as such, it was also assumed that the force was evenly distributed across both connections.
It is acknowledged that once the strength started to degrade in J02 and the movement between the top and bottom connection was not equal, the load would likely not have evenly been shared across both connections. However, the post peak behaviour in this instance is not of great value and so this assumption was deemed appropriate.

A force-displacement response of the individual WSP and GPP connection in J01 and J02 respectively was then developed by assumed 50% of the total force and averaging the top and bottom displacement of each respective set of connections (i.e. the average of L02-SPOT and L03-SPOT). The force-displacement response is shown in Figure 7.55.

![Figure 7.54: Displacement-time response of test specimens J01 (right) and J02 (left).](image)

![Figure 7.55: Force-displacement connection response of test specimen J01 (right) and J02 (left).](image)

### 7.3.4.2 Connection Stiffness

The second important parameter for the performance of a panel-to-panel connection is stiffness. The stiffness of each of the respective connections was determined by taking a secant stiffness from the origin to point corresponding to 70% of the maximum capacity of the connection being reached. A value of 70% was selected because this represents approximately 80% of the design capacity (e.g. the capacity reduction factor for a shear stud in accordance with AS 2327.1 [X48] is 0.85 and $0.8 \times 0.85 \approx 0.7$). It is recommended later in Section 7.4 that precast core connections be capacity designed and their loading limited to 80% of their design capacity.
The stiffness was calculated for the positive and negative direction. The positive and negative direction stiffness values were averaged together and taken as the stiffness for each respective connection type. The stiffness for each connection is summarised in Table 7.12. The process for determining the stiffness for each connection is illustrated in Figure 7.56.

**Table 7.12: Building core connection specimen initial stiffness.**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$k^{+ve}$ (kN/mm)</th>
<th>$k^{-ve}$ (kN/mm)</th>
<th>$k_{ave}$ (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>J01 (WSP)</td>
<td>36.40</td>
<td>52.54</td>
<td>44.47</td>
</tr>
<tr>
<td>J02 (GPP)</td>
<td>48.20</td>
<td>104.4</td>
<td>76.29</td>
</tr>
<tr>
<td>J03* (PTC)</td>
<td>1565</td>
<td>1938</td>
<td>1751</td>
</tr>
</tbody>
</table>

* The positive direction stiffness for test specimen J03 is taken as the initial stiffness of the connection prior to the post tensioning force being developed. After the initial post tensioning force is exceeded, the stiffness drops significantly to 120.9 kN/mm.

The WSP connection had the smallest stiffness of the three connections and was equal to 44.5 kN/mm. The system level test specimen denotes S05 had the same WSP connection and it was shown that it was not ‘stiff enough’ to allow effective composite behaviour to be developed (refer Section 7.2.4.6). Suggesting that 44.5 kN/mm is too flexible for meaningful composite action to be developed.

The first prototype connection, i.e. the GPP, was 73% stiffer than the WSP connection and had a stiffness of 76.3 kN/mm. The GPP connection also had a maximum capacity that was 73% higher than the WSP connection. This suggests the GPP connection is a superior alternative to WSP connections, while providing the additional benefit of avoiding on-site welding. Additional analysis is still required to assess if the higher stiffness of 76.3 kN/mm is sufficient to allow for more effective composite action to be developed.

The second prototype connection, i.e. the PTC, was nearly 40 times stiffer than the WSP connection and had a stiffness of 1750 kN/mm. The very high stiffness and strength of the PTC connection suggest it would be close to or equally as effective as a wet joint in precast building cores, which essentially provides the equivalent performance of a cast in-situ element. It should be noted that once the PTC is loaded in tension to a force exceeding the initial post tensioning force, the stiffness significantly reduces to 121 kN/mm. It is recommended in a practical application that the post tensioning force that is applied would be greater than the required design force. The post tensioning force was limited in the test specimen due to construction constraints in the laboratory.
7.3.4.3 Test Specimen J01 Failure Mechanism

The failure mechanism of test specimen J01 was yielding and fracturing of the shear studs. The maximum loading was governed by the yielding of the studs and then overall failure of the connection was due to fracturing of the studs. The fractured studs of the top cast-in plate on panel A of J01 are shown in Figure 7.57. The specimen was able to develop its maximum load with minimal damage and cracking of each individual wall panel, as shown in Figure 7.58. It was only after the maximum strength of the connection was exceeded and it was subjected to much larger displacement increments that local degradation and failure of the concrete around the connection started to occur. Damage progression photos for test specimen J01 are provided in Appendix D.

![Figure 7.57: Test specimen J01 fractured shear studs.](a) panel A, top cast-in plate (front view)  
(b) panel B, bottom cast-in plate (side view)

![Figure 7.58: Test specimen J01 response at maximum load (bottom connection).](a) load cycle 11, maximum positive load  
(b) load cycle 12, maximum negative load

Vertical cracking of panel B was observed on the top WSP connection in load cycle 11 (i.e. maximum positive load). This suggests that if the horizontal cross tie was not present, tensile splitting along the vertical edge of panel B may have been the critical aspect of the connection. This strongly indicts the importance of having a horizontal cross tie, which would typically be in the form a ‘U’ bar, above and below the shear studs.
The vertical force is transferred between the panels by the stitch plate via the shear studs, which are interlocked with each respective panel. This creates a couple that is equal to the vertical force multiplied by half the distance between the shear studs on each respective panel. The couple is resisted on each side of the connection by opposing horizontal forces on the top and bottom shear stud. The magnitude of the horizontal force that acts on each stud is dependent on the geometry of the connection. The shear studs then have a vertical force applied to them that is equal to half of the vertical force transferred across the connection. The resultant shear force applied to the shear studs can then be calculated. This process is outlined by Equation 7.1 and Figure 7.59.

\[
F_y = F_{y,1} + F_{y,2}
M = F_y \times \left( \frac{170}{2} \right)
F_{y,1} = 0.5F_y
F_{x,1} = \frac{M}{100} = \frac{85F_y}{100} = 0.85F_y
F_{r,1} = \sqrt{(F_{x,1})^2 + (F_{y,1})^2} = \sqrt{(0.85F_y)^2 + (0.5F_y)^2} = 0.99F_y
\]

\[
[F_y]_{max} = 1.01 \times [F_{r,1}]_{max}
\]

Where:
- \([F_y]_{max}\) = maximum vertical shear strength of the connection
- \([F_{r,1}]_{max}\) = maximum shear capacity of the shear stud

![Diagram of load distribution between shear studs](image)

**Figure 7.59:** Welded stitch plate load distribution between shear studs.

The maximum load the WSP connection in test specimen J01 could withstand was 88 kN (Figure 7.55). Using the method outlined above, the resultant shear force acting on each shear stud is then 87 kN. The maximum capacity of 19 mm diameter shear stud is usually determined in accordance with AS 2327.1, which for concrete with a minimum compressive strength of 32 MPa, is taken to be 93 kN. This is somewhat in good agreement with the results of J01. Further, AS 22327.1 stipulates a capacity reduction factor of 0.85 for shear studs, resulting in an ultimate limit capacity of 79 kN for a 19 mm diameter shear stud.
7.3.4.4 Test Specimen J02 Failure Mechanism

The failure mechanism of test specimen J02 was a combination of cyclic degradation of the grouted pocket and failure of the M20 bolts. The cast-in bolts in the grouted pocket (i.e. the 2-M20 grade 8.8 bolts) failed during the test, as shown in Figure 7.60(b), however it is difficult to ascertain with certainty whether the bolts failing or cycle degradation of the grouted pocket was the critical component of the connection governing its capacity. It could be argued that the bolts began to fracture, which caused the softening and loss of strength. The degradation of the grouted pocket would then have just been a result of the large cyclic vertical movements the specimens were subjected to. Equally, it could be argued that the grouted pocket may have locally began to degrade around the bolts, which then meant the shear plane where the load was transferred from the bolts to the grouted pocket shifted further away from the face of the adjacent panel, allowing the bolts to be subjected to combined bending and shear, which would simultaneously result in a strength drop and increase in flexibility. In all likelihood, the failure mechanism was a combination of the two, however it is believed the former was more prominent of the two.

Interestingly, the cast-in ferrules on panel C performed well and were able to transfer the loads to the adjacent panel without causing any bearing failure to the concrete directly surrounding them. This is shown in Figure 7.62(a), which has a photo of the grouted pocket after the testing was completed and the grouted pocket had been locally demolished. This shows that the concrete around the ferrules is in good condition, with no bearing failures observed. The internal threads on each ferrule were also in good condition and it appeared as though no yielding of any ferrule threads occurred.

It is proposed that the failure mechanism of the GPP connection was to be due to local bearing failure of the grout around the cast-in bolts, which resulted in the shear plane shifting away from the interface between the two panels and resulting in the bolts being subjected to bending, as shown in Figure 7.61 and discussed above. Eligehausen, Mallee and Silva [E7] proposes that when cast-in bolts are subjected to bending, Equation 7.2 to determine the maximum shear capacity of the bolt. Equation 7.2 is a function of the moment capacity of the bolt (i.e. $M_{u,s}$), the distance between the support plane and where the load is applied (i.e. $l$) and a coefficient (i.e. $\alpha_m$) that equals 1 for single curvature bending or 2 for double curvature bending.

![Figure 7.60: Test specimen J02 cast-in ferrules and bolts.](image-url)
Eligehausen et al. [E7] cites a study by Scheer, Peil and Nölle [S24] that proposed Equation 7.3 for determining the maximum bending moment in a bolt at failure. Equation 7.3 is a function of the yield stress of the bolt and the section modulus for the thread part of the bolt that is calculated based on the net tensile area. In test specimen J02 M20 bolts were adopted which have a tensile stress area of 245 mm$^2$. That corresponds to a circular bar with an equivalent diameter of 17.7 mm. The section modulus of a 17.7 mm circular bar is 544 mm$^3$. Using Equation 7.3 and the yield stress of the bolts (i.e. Figure 7.47(a)), the moment capacity of the M20 thread is 0.768 kNm.

$$V_{u,s} = \frac{\alpha_m M_{u,s}}{l} \quad \ldots 7.2$$

Where:
- $V_{u,s}$ = shear capacity of bolt at failure
- $\alpha_m$ = 1.0 for single curvature bending
  = 2.0 for double curvature bending
- $M_{u,s}$ = moment capacity of bolt at failure
- $l$ = effective moment-arm length (refer Figure 7.61)

$$M_{u,s} = 1.7 W_{el} f_{sy} \quad \ldots 7.3$$

Where:
- $W_{el}$ = section modulus for the threaded part calculated based on the net tensile area
- $f_{sy}$ = yield stress of the bolt

The distance $l$ in Figure 7.61 can be back calculated by substituting the moment capacity of the M20 bolts calculated above and the maximum shear force determined from the test into Equation 7.2. The maximum shear capacity of J02 was 304 and 308 kN in the positive and negative directions respectively. Using average of these values (i.e. 306 kN) and assuming the force is shared evenly between the two GPP connections and the two bolts in each GPP connection, the
shear capacity of the bolts are 76.5 kN. The distance \( l \) can then be calculated to be 20 mm. This suggests the distance over which bearing failure between the cast-in bolt and grout occurred was 20 mm, which is conveniently also equal to the diameter of the cast-in bolt.

This approach could possibly be extrapolated to calculate the capacity of a higher strength connection in 200 or 250 mm thick panels using larger M24 ferrules and cast-in bolts, where for example the length \( l \) would be 24 mm. However, there are many uncertainties in the process outlined above and as such, additional testing is recommended to develop comprehensive, experimentally confirmed, design rules for the GPP connection. This test however shows strong proof-of-concept for the new prototype GPP connection.

During the initial load cycles of test specimen J02 some minor horizontal cracks developed in the grouted pocket. As the loading cycle displacement increments increased, these cracks widen and really appeared to initiate the cyclic degradation of the grouted pocket. This is shown in Figure 7.63, which shows the cracking that occurred around the bottom GPP connection during the first three series of loading cycles. Figure 7.62 shows when the photos in Figure 7.63 were taken relative to the force-displacement response of the GPP connection in J02. Further, additional damage progression photos, showing the overall specimen, for test specimen J02 are provided in Appendix D.

It is believed that if nominal confinement reinforcement was provided within the grouted pocket, the cyclic degradation of the grout would have been reduced significantly. As a result, this would likely reduce the distance \( l \) calculated previously and allow the connections to resist a higher maximum vertical shear force. Further, the value determined for \( l \) of 20 mm is also the width of the panel gap and the width of grout not confined by the surrounding concrete of panel D (refer Figure 7.61). This could also suggest, as an alternative to what was discussed above, that the length \( l \) was governed by the width of the panel gap, as opposed to the bolt diameter. If so, providing confinement within the grouted pocket would very likely decrease the \( l \) and increase the capacity of the connection. As a minimum though, it would almost certainly increase the overall robustness of the connection and prevent the significant strength degradation that occurred between loading cycles in the same loading series, e.g. the strength reduction seen between load cycles 31 and 32 and their subsequent positive and negative loading cycle respectively (refer Figure 7.62). The nominal confinement reinforcement recommended would be 6 mm round bar ligatures in a helical arrangement around the bolts, similar to what is used to confine the 'live end' of post tensioning bars and tendons.

![Figure 7.62: Test specimen J02 reference points for damage progression photos.](image)
Alternately, fibre reinforced grout could possibly be used instead of cementitious grout and the proposed helical confinement reinforcement. It is possible that fibre reinforced grout may present additional construction difficulties as the fibres may prevent the grout from being pumped as easily as conventional contentious grout, meaning its use could result in added difficulties on-site while grouting the panel pocket. However, this is still recommended as a promising area for further research.

Further, Eligehausen et al. [E7] proposes Equations 7.4 and 7.5 for the maximum capacity of a cast-in anchor that fails due to steel rupture and spalling of concrete respectively. Using these equations and assuming the grout is well confined, the maximum strength of the bolt on the grouted pocket side of the connection is 124.5 kN and is governed by steel rupture.

Figure 7.63: Test specimen J02 damage progression of bottom connection.
This strength was calculated using the core area of an M20 bolt, which is 225 mm², the ultimate tensile stress of the bolt (refer Figure 7.47(a)), the modulus of elasticity of the grout, which is 25,000 MPa according to the manufacturer’s technical datasheet, and the compressive strength of the grout, which was 70 MPa. The strength on the panel C side of the connection is likely to be governed by steel rupture, meaning it will also be 124.5 kN, as the ferrule increases the bearing area of the bolt against the concrete meaning concrete spalling failure is unlikely. The failure load of the test was 76.5 kN per bolt, which means that if one of the above methods for confining the grout was successful (i.e. helical confinement ligatures or fibre reinforced grout), the strength of the connection could possibly be increased by up to 60% to around 125 kN per bolt or 250 kN per GPP connection.

\[
V_{u,s} = \alpha A_s f_{su} \quad \ldots 7.4
\]

Where: \( \alpha = 0.6 \)

\( A_s = \text{bolt area} \)

\( f_{su} = \text{ultimate tensile stress of the bolt} \)

\[
V_{u,sp} = 0.5A_s \sqrt{E_c f_c} \quad \ldots 7.5
\]

Where: \( A_s = \text{bolt area} \)

\( E_c = \text{modulus of elasticity of concrete} \)

\( f_c = \text{compressive strength of concrete} \)

### 7.3.4.5 Test Specimen J03 Failure Mechanism

The failure mechanism of test specimen J03 in the positive direction was due to inelastic strain hardening and fracturing of the Macalloy bar and in the negative direction was believed to be due to cyclic degradation of the grout pad at the corbel interface. The behaviour of J03 was largely in accordance with well-established structural engineering principles and serves as proof-of-concept for this connection. Damage progression photos of test specimen J03 are provided in Appendix D.

### 7.3.5 Blind Prediction Study of Welded Stitch Plate Connections

A blind study assessing the strength of the WSP connection used in test specimen J01 was undertaken with multiple structural engineering design consultancies in Australia. The blind study highlighted that there was a wide variation of opinion of how to calculate the capacity of the connection. The majority of designers thought the shear studs would be the critical component of the connection, however amongst them, there was considerable variation regarding how they calculated the loading on the shear studs. Typically, they fell into three groups: the first group assumed a moment was developed locally across the connection equal to the vertical shear force applied to the connection multiplied by half the distance between the shear studs on each adjacent panel (i.e. a double curvature moment gradient was assumed across the stitch plate, as shown in Figure 7.59); the second group also assumed a moment was developed locally across the connection, however they said it was equal to the vertical shear force multiplied by the total distance between the shear studs on each adjacent panel (i.e. a single curvature moment gradient was assumed across the stitch plate); and the third group assumed no moment was developed locally across the connection. As such, each of these groups had considerably different loading scenarios and design actions for the shear studs.
The three different loading scenarios result in different connection strengths as follows: the group one approach results in the maximum shear capacity of the connection being equal to 1.01 times the shear capacity of a single shear stud (as shown in Figure 7.59); the group two approach results in the maximum shear capacity of the connection being equal to 0.56 times the shear capacity of a single shear stud; and the group three approach results in the maximum shear capacity of the connection being equal to 2 times the shear capacity of a single shear stud. This meant the submissions for the strength of the connection, where it was assumed the shear studs were the critical component, ranged from about 52 to 186 kN (assuming a capacity reduction factor of 1.0). While the majority of participants thought the shear studs would be critical, there were also submissions that thought the weld strength would be the critical component and, in this instance, the capacity was thought to be about 230 kN.

It is clear from the results of that the group one assumption is the most appropriate design model for the connection. Forty per cent of the participants assumed this design procedure, meaning more than half the participants from industry incorrectly calculated the capacity of this connection, despite it being a widely used ‘industry standard’ style of connection.

### 7.3.6 Conclusions and Recommendations

The following conclusions and recommendations have been made following the component level precast building core connection experimental testing program:

1. The first new innovative prototype connection, i.e. the ‘grouted panel pocket’ (GPP) connection, was 1.73 times stronger and 1.73 times stiffer than the baseline WSP connection. The test provides proof-of-concept that the GPP connection is potentially a viable substitute for WSP connections in jointed precast building core.

2. The second new innovative prototype connection, i.e. the ‘post tensioned corbel’ (PTC) connection had a much greater strength and stiffness than the WSP and GPP connections. The stiffness of the PTC connection was greater than 40 times that of the WSP and GPP connections. The test provides proof-of-concept that the PTC is potentially a viable substitute for wet joints in jointed precast building cores.

3. While test specimen J02 provided proof-of-concept for the GPP connection, more testing would be required to establish comprehensive design rules for this type of connection to allow for widespread industry adoption.

4. It is believed that the capacity of the GPP connection could be substantially increased by providing confinement reinforcement in the grouted pocket. This could be performed by proving 6 mm diameter helical ligatures or fibre reinforced grout. Further research is recommended.

5. The PTC connection appears to be a very effective method for constructing jointed precast cores. Its design would be based on well-established structural engineering principles and could be adopted in industry immediately.

6. A blind prediction study was performed with multiple structural engineering design consultancies in Australia to assess the strength of the WSP connection used in this experimental study. The study showed that there is widespread opinion on how to calculate the capacity of these connections, which resulted in participants having submissions that varied by a factor of nearly five. Approximately 40% of the participants calculated the strength of the connection to within 10% of the actual failure load of the test specimen. However, other participants calculated the strength to be from 0.55 times weaker up to 2.3 times stronger than the actual capacity.
7.4 Proposed Precast Building Core Connections

This section proposes a series ‘standard’ WSP, GPP and PTC core connections for precast concrete building cores based on the results of the component level testing outlined in Section 7.3.

7.4.1 General Design Procedure

Horizontal panel-to-panel connections joining adjacent precast panels are typically required to predominantly transfer vertical shear forces, allowing the individual panels to act as one composite cross-section. These connections can be used to connect panels that are both parallel and perpendicular to each other, as shown in Figures 7.64(a) and 7.64(b) respectively.

![Figure 7.64: Different panel-to-panel connection configurations.](image)

When designing these connections for precast building cores under seismic actions, it is being recommended that the connections be ‘capacity designed’ to ensure their response remains linear elastic. This means the design vertical shear forces the connections are designed for would be calculated using the equivalent linear elastic system (i.e. $\Omega = 1.0$ and $\mu = 1.0$) or taken as the shear force required to allow the mean maximum bending moment of the core to be developed, whichever is less. Further, it also being recommended to limit the loading on the connection to 80% of the ultimate capacity, to further ensure a linear elastic response is maintained.

The following three sub-sections, i.e. Sections 7.4.2, 7.4.3 and 7.4.4, will present a set of proposed details for WSP, GPP and PTC connections respectively. It is being recommended that these connections are designed such that the failure is restricted to the shear studs, cast-in bolts and PT bar for the WSP, GPP and PTC connections respectively. Meaning, if the connection is subjected to an overload scenario, a sudden brittle failure will not occur, as each of these fail mechanisms were shown to be able to undergo large amounts of displacement prior to ‘complete failure’ and losing all vertical shear resistance. A maximum shear strength and vertical stiffness is provided for each connection. Each connection has been developed such that a clear cover of 30 mm can be achieved.

7.4.2 Welded Stitch Plate Connections

A series of proposed WSP connections are presented in Figure 7.65, which include two, three, four and six shear stud options. The strength and stiffness of each WSP configuration is presented in Table 7.13. The proposed WSP connections are for perpendicular panels (i.e. Figure 7.64(a)), however they could equally be adopted for parallel panel connections (i.e. Figure 7.64(b)) by simply mirroring the left half of the connection. The connection strengths given in Table 7.13 are based on the shear studs being the critical component of the connection and as such, are the forces required to develop the ultimate shear strength in the critical shear stud for each respective stud configuration, which was 79 kN (i.e. $\phi V_s = 0.85 \times 93 = 79$ kN), in accordance with AS 2327.1. The procedure to calculate the individual stud loading is presented in Appendix D, in addition to design guidance for calculating the weld and stitch plate strength required.
Figure 7.65: Proposed welded stitch plate details.
Table 7.13: Proposed welded stitch plate connection properties.

<table>
<thead>
<tr>
<th>Number of shear studs</th>
<th>Ultimate shear strength, i.e. $\phi V_u$</th>
<th>Vertical stiffness, i.e. $k_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>89.0 kN</td>
<td>54.9 kN/mm</td>
</tr>
<tr>
<td>3</td>
<td>159.4 kN</td>
<td>143.7 kN/mm</td>
</tr>
<tr>
<td>4</td>
<td>152.0 kN</td>
<td>148.7 kN/mm</td>
</tr>
<tr>
<td>6</td>
<td>281.4 kN</td>
<td>329.0 kN/mm</td>
</tr>
</tbody>
</table>

The vertical connection stiffness (i.e. $k_v$) of the different stud configurations was determined by first determining the individual stud stiffness (i.e. $k_x$) using the results of test specimen J01. It was discussed further in Section 7.2.4.2 and shown that the displacement of the stitch plate was insignificant compared to the rigid body movements of the whole connection. Meaning the movement of the WSP connection in J01 was essentially from rigid body translation and rotation of the whole connection, which was due to movement of the shear studs in the concrete. By assuming all the movement in the connection was due to rigid body movement the shear stud stiffness can be calculated to be proportionate to the vertical connection stiffness, refer Figure 7.66 Equation 2, which allows the shear stud stiffness to be calculated as 173.1 kN/mm, based on an overall vertical connection stiffness of 44.5 kN/mm for the WSP connection in test specimen J01 (refer Table 7.12 and Figure 7.56).

![Figure 7.66: Calculation of shear stud stiffness (test specimen J01 shear stud configuration).]  

Using the individual stud stiffness calculated above (i.e. $k_x = 173.1$ kN/mm), the overall vertical connection stiffness for the different stud configurations shown in Figure 7.65 can be calculated using the reverse process as to what is described in Figure 7.66, which is as follows:

1. Calculate for the proportion of vertical and horizontal force each shear stud is resisting.  
   This is calculated in the same manner to which the loading on individual bolts in bolt-groups in structural steel connections is performed. This is also shown in Appendix D.
2. Calculate the vertical and horizontal movement of each shear stud using the individual shear stud stiffness of 173.1 kN/mm and a unit load (i.e. $F = 1$ kN).
3. Calculate the vertical movement in the middle of the stitch plate (i.e. $0.5\Delta_y$) caused by rigid body movement of the individual shear studs calculated in Step 2.

4. Calculate the vertical connection stiffness using Equation 1 in Figure 7.66, the vertical movement calculated in Step 3 and the unit load (i.e. $F = 1\, \text{kN}$).

An example of the four-stud free body diagram, similar to the two-stud diagram shown in Figure 7.66, is presented in Figure 7.67 to further illustrate and describe the four-step procedure outlined above.

![Figure 7.67: Calculation of vertical connection stiffness for the four-stud configuration.](image)

The WSP details in Figure 7.65 also include an additional 30 mm recess of the cast-in plates into each respective panel. The cast-in plate is recessed into the panel to allow the connection to be infilled with a non-structural grout for fire protection purpose. The cast-in plates are almost always recessed for this purpose, as identified in Section 3.3.2 and Figure 3.11.

The cast-in plates for the WSP details in test specimen J01 were not recessed due to size restrictions in both the test machine and laboratory. Test specimen J01 was meant to represent a component level connection test of the system level test specimen denoted S05. The maximum panel thickness for specimen S05 was 150 mm to ensure the specimen would not be too heavy to lift using the overhead gantry crane in the SSL and additionally, it wasn’t ‘too strong’ to test in the MAST System.

It is believed that further recessing the cast-in plates into the panel will not increase the overall strength of the connection and will have minimal impact on the connections stiffness, because the cast-in plate will effectively still be within the cover concrete region of the panel. Further, when welding the stitch plates to the cast-in plates on site, the cast-in plates temporary expands due to the high heat that is transferred into the plate from welding. This expansion causes the plate to bear against the surrounding cover concrete and in some cases, significantly weaken the cover concrete and initiate spalling, which was observed in some of the connections in test specimen S05 and J01. After the welding is completed and the cast-in plate begins to cool, it will contract. It is believed this process creates enough of a gap, albeit very small (i.e. less than 0.5 mm) that when the WSP connection is loaded, the shear studs are providing all the restraint against rotation. However, more testing on thicker 200 and 220 mm RC panels with recessed cast-in plates is recommended to confirm the stiffness and strength properties proposed in Table 7.13.
7.4.3 Grouted Panel Pocket Connections

Two proposed GPP connections are presented in Figure 7.68, which include three and six cast-in-bolt configurations. The strength and stiffness of each GPP connection is presented in Table 7.14. The proposed GPP connections in Figure 7.68 are for perpendicular panels (i.e. Figure 7.64(a)). A detail for a proposed three bolt GPP connection for parallel panels (i.e. Figure 7.64(b)) is presented in Figure 7.69.

<table>
<thead>
<tr>
<th>Number of cast-in bolts</th>
<th>Ultimate shear strength, i.e. $\phi V_u$</th>
<th>Vertical stiffness, i.e. $k_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>135 kN</td>
<td>115 kN/mm</td>
</tr>
<tr>
<td>6</td>
<td>270 kN</td>
<td>230 kN/mm</td>
</tr>
</tbody>
</table>

The strength of the GPP connection in test specimen J02, which was detailed using two M20 cast-in bolts, was about 150 kN or 75 kN per bolt. The capacity of the proposed connections is being taken as 75 kN times the number of bolts in the detail, therefore, the three and six bolt detail has a capacity of 225 kN and 450 kN respectively. Due to the uncertainty into the exact failure mechanism of this connection, as discussed in detail in Section 7.3.4.4, a conservative capacity reduction factor (i.e. $\phi$) of 0.6 is being recommended. This means the ultimate strength of the three and six bolt connection is being proposed as 135 and 270 kN respectively, as summarised in Table 7.14.

The stiffness of the proposed GPP connections are simply being taken as 1.5 and 2.0 times the stiffness of the GPP connection in test specimen J02 for the three and six bolt connections respectively, given the three and six bolt details have 1.5 and 2.0 times the number of bolts than J02. This means the vertical stiffness of the three and six bolt connection is being proposed as 115 and 230 kN/mm respectively, as summarised in Table 7.14.

The two details proposed in Figure 7.68 use M20 bolts, matching what was adopted in the GPP connection for test specimen J02. It is likely that M24 bolts would provide a significant increase in capacity, however given the uncertainties associated with the exact failure mechanism, as discussed above, the proposed details use M20 bolts. Further testing is recommended to allow the development of higher capacity connections using M24 bolts.

The connection in test specimen J02 adopted double-ended ferrules with a 400 mm long bolt cast-in to the rear side of the ferrule, whereas in the proposed details an elephant foot ferrule is used. It is believed that the ferrules were subject to minimal direct tensile stresses and resisted the vertical shear forces in combined shear and bending. Hence, pull-out of the ferrule should not be a problem. Further, the proposed details have two vertical bars either side of the ferrules, with horizontal lapped ‘U’ bars above and below each ferrule, meaning that if a pull-out scenario was to develop, the concrete failure cone the ferrules would mobilise is restrained by this reinforcement. The primary reason for not recommending a double-ended ferrule detail with the cast-in bolt, as per test specimen J02, is that it presents congestion problems with the grout tube connection if the GPP connections are located near the top or bottom of the panels.
Figure 7.68: Proposed grouted panel pocket details.
7.4.4 Post Tensioned Corbel Connections

Three proposed PTC connections are presented in Figure 7.70, which include corbel details for both one and four post tensioning bar configurations. The strengths of the three details are summarised in Table 7.15. The stiffness of the PTC connection is significantly higher than the other connections (refer Section 7.3.4.2) and believed to be stiff enough to be the equivalent of a ‘wet joint’ (i.e. Figure 3.11(b)), meaning full composite action can be assumed, i.e. infinitely rigid connection. The maximum strength of the PTC connection is taken as the initial post tensioning (PT) load because once the initial PT load is exceeded, the stiffness of the connection significantly reduces (i.e. Table 7.12 and Figure 7.56) and it will most likely lose its ability to allow the individual panels to act as one composite section. The post tensioning loads in Table 7.15 equal approximately 70% of characteristic maximum load of the bar. It should be noted that the PT should occur prior to grouting the grout tube connections at the base of the panel.

Table 7.15: Proposed post tensioned corbel connection properties.

<table>
<thead>
<tr>
<th>Connection</th>
<th>Post tensioning load per bar</th>
<th>Ultimate shear strength, i.e. $\phi V_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>200 mm thick panel, 1x 32Ø Macalloy bar</td>
<td>550 kN</td>
<td>550 kN</td>
</tr>
<tr>
<td>250 mm thick panel, 1x 36Ø Macalloy bar</td>
<td>700 kN</td>
<td>700 kN</td>
</tr>
<tr>
<td>250 mm thick panel, 4x 26.5Ø Macalloy bar</td>
<td>375 kN</td>
<td>1500 kN</td>
</tr>
</tbody>
</table>
Figure 7.70: Proposed post tension corbel details.
7.5 Conclusions

This chapter has presented the details and results of the third experimental testing program. The testing consisted of three large-scale system level tests and three component level tests. The system levels tests were single-storey jointed precast concrete building cores that were meant to represent the ground floor component of a taller four-storey wall. The three component level specimens consisted of one baseline specimen that had ‘welded stitch plate’ (WSP) connections and two other specimens that were connected using two new innovative prototype connections. The first prototype connection is called a ‘grouted panel pocket’ (GPP) connection and the second a ‘post tensioned corbel’ (PTC) connection. A summary of conclusions for the system level tests and component levels tests is presented at the end of each respective sub-section, i.e. Section 7.2.5 and 7.3.6 respectively. The major conclusions and recommendations are restated below.

System level precast concrete building core testing:

1. Despite the lateral strength decreasing significantly, often below 20% of the maximum response, the walls could withstand very large in-plane lateral drifts prior to axial load failure occurring (i.e. complete structural collapse). Test specimens S03, S04 and S05 were loaded to 6.5%, 8.0% and 4.9% drift respectively without axial load failure occurring, after which the test was terminated.

2. The WSP connections were not stiff enough to allow full composite action to be developed. This resulted in the effective moment of inertia of test specimen S05 being 25% lower than the cast in-situ building core specimen tested in Chapter 6 (i.e. S02). Further, it also meant that only 80% of the theoretical maximum moment capacity of the wall could be developed before flexure failure occurred via fracturing of the vertical reinforcement.

3. Low ductility reinforcement allows only marginal ductility to be developed before fracturing of the reinforcement occurs and results in a sudden reduction in lateral strength.

Component level precast concrete building core connection testing:

1. The first new innovative prototype connection, i.e. the ‘grouted panel pocket’ (GPP) connection, was 1.73 times stronger and 1.73 times stiffer than the baseline WSP connection. The test provides proof-of-concept that the GPP connection is potentially a viable substitute for WSP connections in jointed precast building cores. However, more testing would be required to establish comprehensive design rules for this type of connection to allow for widespread industry adoption.

2. The second new innovative prototype connection, i.e. the ‘post tensioned corbel’ (PTC) connection had a much greater strength and stiffness than the WSP and GPP connections. The stiffness of the PTC connection was greater than 40 times that of the WSP and GPP connections. The test provides proof-of-concept that the PTC is potentially a viable substitute for wet joints in jointed precast building cores.

3. A blind prediction study was performed with multiple structural engineering design consultancies in Australia to assess the strength of a typical WSP connection. The study showed that there is widespread opinion on how to calculate the capacity of these connections, with predictions varying by almost a factor of five.

4. The chapter is concluded by providing a series of recommended details for WSP, GPP and PTC connections. Advice is also provided on the ultimate vertical shear strength and vertical stiffness of each connection.
Chapter 8

Modelling the Force-Displacement Behaviour of RC Walls

Displacement-based seismic design procedures require knowledge of the force-displacement behaviour of both the overall building and individual lateral load resisting elements, i.e. walls or building cores, which has been the focus of this thesis. This chapter will initially focus on the development and validation of a user-friendly and transparent analysis program for predicting the back-bone force-displacement behaviour of slender (i.e. flexure controlled) RC walls and building cores, which were shown in Chapter 3 to be the dominant form of wall construction in Australia. The program, which is called WHAM, is written using Microsoft Excel spreadsheets and the intent is to release the program free-of-charge as a design tool to assist structural engineers or be used as an educational tool for students or researchers. The chapter is then concluded with the results of a parametric study using WHAM, which has allowed the development of a series empirical models for estimating a bilinear moment-curvature response curve for both limited ductile RC walls and building cores.

8.1 Development of a User-Friendly and Transparent Analysis Program for Predicting the Force-Displacement Behaviour of RC Walls

A simple-to-use and transparent analysis program was developed using Microsoft Excel spreadsheets for predicting the force-displacement behaviour of RC walls and building cores. The program is called WHAM and was developed using Excel spreadsheets as they offer complete transparency, such that the user can easily examine and understand how the program works, while also being able to easily further develop or expand the capabilities of the program to suit their respective needs. One of the primary objectives while developing WHAM was to ensure it was simple-to-use and had a user-friendly interface so designers or students, which whom have had little or no exposure and experience using non-linear analysis packages, could easily understand, adopt and use the program.
WHAM is a fibre-element analysis program (as outlined in Section 4.3.2), which determines the moment-curvature response of the section based on the concrete and reinforcement non-linear stress-strain material models selected and the axial load applied to the wall. The fibre-element analysis procedure outlined in Section 4.3.2 was modified to account for tension stiffening of the section, as proposed in Section 5.6 and illustrated diagrammatically in Figure 5.31. The tension stiffening modifications in WHAM are presented later in Section 8.1.3.

WHAM, after calculating the moment-curvature response, then calculates the force-displacement response of the wall using Equations 8.1 to 8.6, which assumes an equivalent plastic hinge at the base of the wall and a linear curvature profile up the height of wall. The plastic hinge model adopted in WHAM is the Priestley et al. [P4] model, which was previously presented in Section 4.3.3. An idealised force-displacement response is presented in Figure 8.1.

Cracking:

\[ \Delta_{cr} = \frac{\phi_{cr} H_{eff}^2}{3} \quad \ldots 8.1 \]

\[ F_{cr} = \frac{M_{cr}}{H_{eff}} \quad \ldots 8.2 \]

Point of first yield:

\[ \Delta'_y = \frac{\phi_{y}' H_{eff}^2}{3} \quad \ldots 8.3 \]

\[ F_{y} = \frac{M_{y}}{H_{eff}} \quad \ldots 8.4 \]

The i-th point after first yield:

\[ \Delta_i = \Delta'_y \left( \frac{M_i}{M_y} \right) + \left[ \phi_i - \phi_{y}' \left( \frac{M_i}{M_y} \right) \right] L_p \left( H_{eff} - \left( \frac{L_p}{2} - L_{sp} \right) \right) \quad \ldots 8.5 \]

\[ F_i = \frac{M_i}{H_{eff}} \quad \ldots 8.6 \]

Figure 8.1: Idealised force-displacement response.
Equations 8.1 to 8.6 calculate the force-displacement response of the wall at the effective height of the wall (i.e. $H_{\text{eff}}$). Similarly, the displacement at various storey's can also be calculated using Equations 8.7 to 8.9.

**Cracking:**

$$\Delta_{n,\text{cr}} = \frac{\phi_{cr} h_n^2}{2} \left(1 - \frac{h_n}{3H_{\text{eff}}} \right) \quad \cdots 8.7$$

**Point of first yield:**

$$\Delta'_{n,y} = \frac{\phi'_{y} h_n^2}{2} \left(1 - \frac{h_n}{3H_{\text{eff}}} \right) \quad \cdots 8.8$$

**The i-th point after first yield:**

$$\Delta_{n,i} = \Delta'_{n,y} \left(\frac{M_i}{M_y}\right) + \left[\phi_i - \phi'_y \left(\frac{M_i}{M_y}\right)\right] L_p \left[h_n - \left(\frac{L_p}{2} - L_{sp}\right)\right] \quad \cdots 8.9$$

Where:

- $\Delta_{n,\text{cr}}$ = cracking displacement at the n-th storey
- $\Delta'_{n,y}$ = first yield displacement at the n-th storey
- $\Delta_{n,i}$ = n-th story displacement for the i-th loading point
- $h_n$ = height from the ground to the n-th storey

The program interface of WHAM, the concrete and reinforcement material models and the tension stiffening model integrated into the program, are each presented in the following three subsections, i.e. Sections 8.1.1, 8.1.2 and 8.1.3 respectively. Following this: theoretical and experimental validations of WHAM are presented in Section 8.2; the limitations of the program and proposed future improvements are presented in Section 8.3; and a parametric study using WHAM to study the force-displacement behaviour of limited ductile RC walls and building cores is presented in Section 8.4.

### 8.1.1 Program Interface

The program is split across seven worksheets. The first worksheet is the first of two section input pages, where the user enters the cross section of the wall or building core by entering the x and y nodal coordinates of the section, as shown in Figure 8.2. The program can handle various wall cross sections, e.g. Figures 8.4(a) to 8.4(e), however the program can only handle a maximum of 100 node points. Openings or 'breaks' in the wall (as shown in Figure 8.4(e)) can also be modelled by allowing an 'empty' row in the node input table. The reinforcement in the wall can be generated automatically by entering a maximum reinforcement centre-to-centre spacing or desired reinforcement ratio (i.e. $p_y = A_{sv}/A_y$). The automatic function can also be disabled and the user can input each individual reinforcing bar using the x and y coordinates of each respective bar. Further, the automatic and manual functions can also be used together. The second worksheet is the second of the two section input pages, where the user enters the coordinates for any confined regions within the cross section of the wall, as shown in Figure 8.3 (e.g. the corner regions for the example building core in the figure).
Figure 8.2: WHAM section input page – general cross section.

Figure 8.3: WHAM section input page – confined regions.

(a) rectangular  (b) flanged wall  (c) ’C’ section  (d) Bundled box  (e) Geometric

Figure 8.4: Example wall and building core cross sections that can be analysed in WHAM.
The third and fourth worksheets are the concrete and reinforcement stress-strain material model pages respectively (refer Section 8.1.2) and the fifth worksheet is the tension stiffening model page (refer Section 8.1.3). The sixth page is used to perform the fibre element analysis of the wall to determine the moment-curvature response of the cross section (refer Figure 8.5), which is calculated using a Visual Basic for Applications (VBA) macro. The seventh page is used to calculate the force-displacement response of the wall using the plastic hinge model presented above (i.e. Equations 8.1 to 8.6).

![Fibre Element Analysis](image)

**Figure 8.5:** WHAM moment-curvature results page.

The overall framework of program was initially based on the methodology presented by Lam et al. [L12]. The approach used to divide the complex non-rectangular cross sections into finite constant thickness fibres and to automatically generate reinforcement was adopted from Lam et al. [L12], however a different approach was developed for performing the required iterations to solve each respective point on the moment-curvature response curve.

### 8.1.2 Material Models

The program allows the user to select a predefined concrete and reinforcement stress-strain material model or manually enter stress-strain curves for each respective material. The predefined concrete models include the confined and unconfined Modified Kent and Park [S14], Mander et al. [M23] and Karthik and Mander [K3] models and the unconfined AS 3600 Supp1 [X36] model, which are all further defined in Section 4.4. The recommended concrete stress-strain material model is the Mander et al. [M23] model, which has been shown to provide very good correlation with experimental testing, as presented later in Section 8.2.2.

It is also being recommend for limited ductile walls that the confined Mander et al. [M23] model with a nominal 0.3 MPa lateral confining pressure be used for the core region of the wall, if the horizontal reinforcement spacing is approximately equal to the thickness of the wall and lapped ‘U’ bars, hooked bars or closed ligatures are specified at the end regions of the wall, as shown in Figure 8.7. The nominal 0.3 MPa lateral confining pressure provides minimal strength increase to the concrete, however it greatly increases the relative magnitude of the compressive strains on the descending ‘softening’ branch of the stress-strain curve, as shown in Figure 8.6.
It was observed while experimentally validating the program that by allowing this nominal amount of confinement to the core region of the wall in the limited ductile specimens, the resulting larger strains in the softening branch of the stress-strain curve, resulted in a much better match between the theoretical back-bone curve predicted by the program and the experimental test data. For comparison, well detailed walls with ductile end region confinement steel requirements would normally have a lateral confining pressure around 1.5 to 3.0 MPa.

![Stress-Strain Curve](image)

**Figure 8.6:** Mander et al. [M23] model assuming nominal 0.3 MPa lateral confining pressure.

![Limited Ductile Reinforcement Configurations](image)

**Figure 8.7:** Typical limited ductile horizontal reinforcement configurations.

The predefined reinforcement models include the bilinear and Priestley et al. [P4] models, which are both further defined in Section 4.4. The user can also enter a custom reinforcement model by enter individual stress-strain values.

### 8.1.3 Tension Stiffening Model

The program allows for tension stiffening using the model outlined in Section 5.6 and it can easily be turned off or on in the program prior to undertaking the moment-curvature analysis. When the tension stiffening function is turned off, the program solves the moment-curvature response of the section using the procedure outlined in Section 4.3.2 and illustrated in Figure 4.6. Whereas when the tension stiffening function is turned on, the program performs a modified version of that procedure, where the tension strain at each reinforcing bar is increased from the average global strain to a local reinforcement strain using the tension stiffening modelling (i.e. average global strain to local reinforcement strain relationship) for the specimen, as illustrated in Figure 8.8. The tension stiffening model is dependent on the ratio $k_1$ which is equal to the ratio of vertical reinforcement divided by the ultimate tensile stress of the concrete.

To illustrate how tension stiffening affects the moment-curvature response of wall, a series of analyses were performed on a 2 m long wall, which was 200 mm thick, constructed using 40 MPa concrete and had an axial load ratio of 5%. The results are presented in Figure 8.9.
It can be seen in Figure 8.9 that the tension stiffening behaviour affects the response of walls with lower percentages of vertical reinforcement (i.e. Figure 8.9(a)) much more than walls with higher percentages (i.e. Figure 8.9(b)). The effective moment of inertia for the wall with 0.5% vertical reinforcement (i.e. Figure 8.9(a)) increased by a factor of 1.44 when tension stiffening was considered, whereas the wall with 2.0% vertical reinforcement had an increase of just 1.08. Further, the maximum moment capacity of the wall with 0.5% vertical reinforcement increased relatively more than the wall with 2.0% vertical reinforcement, however even in the case of the former, the increase was quite nominal and only equal to a 4% increase. The tension stiffening behaviour effects the wall with less vertical reinforcement significantly more since the lower percentage of reinforcement results in wider crack spacings, which then means the local strain in the reinforcement is relatively much larger than the average global strain of the concrete. The tension stiffening behaviour in RC elements is discussed in greater detail in Chapter 5.

Figure 8.9: Moment-curvature response for 2 m long x 200 mm thick RC wall constructed with 40 MPa concrete and an axial load ratio of 5%.
8.2 Theoretical and Experimental Program Validation

Two approaches were adopted for validating the program. The first approach was a theoretical validation, which was being used to confirm that both the general coding and fibre element analysis engine worked and was written correctly. The theoretical approach was performed by comparing the moment-curvature results obtained from WHAM against the results from two independent software packages for two different wall cross sections.

The second approach was an experimental validation, which was being used to confirm that the overall process resulted in back-bone force-displacement curves that correlated well with experimentally tested laboratory specimens of RC walls. The experimental approach was performed using the results of 16 test specimens, which included the two cast in-situ wall specimens presented in Chapter 6 and another 14 test specimens from literature. The theoretical and experimental validation approaches are presented in the following two sub-sections, i.e. Section 8.2.1 and 8.2.2 respectively.

8.2.1 Theoretical Validation

The theoretical validation was performed using two different software packages. The first was the commercial software package RAPT [P19], which is a widely used structural analysis package for analysing conventional reinforced, prestressed and post-tensioned elements. The second was an analysis package called Response-2000 [B9], which is a sectional analysis program developed by researchers at the University of Toronto and is available to download free-of-charge online.

The validation was performed on two different wall sections. The first was a rectangular wall section that was 2m long and 250 mm thick and the second was a core wall section that was 2.2 m long, had 1 m wide flanges and wall thicknesses of 200 mm. Both walls were assumed to be constructed using 40 MPa concrete and are both illustrated in Figure 8.10 below.

![Figure 8.10: Walls used to validate the analysis program against RAPT and Response-2000.](image)

8.2.1.1 RAPT Validation

RAPT is a commercial software package that can be used to calculate the interaction diagram (i.e. axial load vs. moment capacity) of rectangular, non-rectangular and circular wall or column sections. RAPT is primarily a force-based structural design tool and as such, it calculates the maximum capacity of elements in accordance with a desired concrete standard or code, which includes AS 3600 [X6], NZS 3101 [X8], ACI 318 [X9] or Eurocode 2 [X10]. This means RAPT uses characteristic material strengths and idealised material stress-strain models, as opposed to non-linear material models that represent the actual in-situ material behaviour.
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RAPT adopts a simplified concrete stress-strain curve, which uses a parabolic curve with an initial slope equal to the young's modulus of the concrete (i.e. $E_c$) until it reaches a maximum and then constant compressive strength of $0.85f'_c$ at a compressive strain denoted $\varepsilon_{co}$. The stress-strain curve can be described using Equations 8.10 and 8.11 below. RAPT uses the Eurocode 2 recommendations for $\varepsilon_{co}$, which for 40 MPa concrete is equal to 0.002. The concrete stress-strain curve adopted by RAPT for 40 MPa concrete is shown in Figure 8.11(a).

\[
\sigma_c = \left[ \frac{0.85f'_c - E_c\varepsilon_{co}}{\varepsilon_{co}} \right] \varepsilon_c^2 + E_c\varepsilon_c \quad \text{where: } 0 \leq \varepsilon_c \leq \varepsilon_{co} \quad \ldots \text{8.10}
\]

\[
\sigma_c = 0.85f'_c \quad \text{where: } \varepsilon_{co} < \varepsilon_c \leq 0.003 \quad \ldots \text{8.11}
\]

Where: $\varepsilon_{co} =$ strain corresponding to the maximum compression strength being reached

$E_c =$ young’s modulus of concrete

RAPT adopts a perfectly elastic-plastic reinforcement stress-strain curve, which ignores any strain hardening of the reinforcement. For D500N reinforcement, which is the most common grade of reinforcement used in Australia, the stress-strain curve has a slope of 200,000 MPa (i.e. $E_s = 200,000$ MPa) until the characteristic yield stress of 500 MPa is reached and then it is constant up to the characteristic ultimate strain of 0.05. The reinforcement stress-strain curve used by RAPT is shown in Figure 8.11(b).

![Figure 8.11: Stress-strain relationship for RAPT validation.](image)

The rectangular and core wall sections were analysed in WHAM using the concrete and reinforcement stress-strain curves presented in Figure 8.11. Each wall section was analysed for incrementally increasing axial load values and the maximum moment capacity for each respective axial load was recorded to construct an interaction diagram for each wall section. The integration diagrams determined from WHAM were then overlayed on the respective RAPT interaction diagrams, as presented in Figure 8.12, where it shown that very good correlation between the two programs was achieved. The corresponding curvatures and neutral axis depths for each maximum moment calculated in WHAM also correlated very well to the RAPT values, with the same level of accuracy to what is shown in Figure 8.12. This shows the general fibre element analysis engine written for WHAM correctly calculates the moment-curvature response for a given wall section and the respective stress-strain material models that have been adopted.
8.2.1.2 Response-2000 Validation

Response-2000 is a sectional analysis program developed by researchers at the University of Toronto and is capable of calculating the moment-curvature response of conventional reinforced and prestressed RC elements. The moment-curvature response of the rectangular wall and core wall sections shown in Figure 8.10 was determined using Response-2000 for axial load ratios of 5%, 10%, 15% and 20%. The response was calculated using the Mander et al. [M23] unconfined concrete stress-strain relationship (i.e. Figure 8.13(a)) and the Priestley et al. [P4] reinforcement stress-strain relationship with no yield plateau and the mean reinforcement material properties for D500N reinforcement proposed in Section 4.4.3 (i.e. Figure 8.13(b)). The moment-curvature response was similarly then calculated using WHAM and compared with the results of Response-2000 (refer Figure 8.14).

It can be seen in Figure 8.14 that Response-2000 and WHAM result in slightly different moment-curvature responses. This however, is due to the tension stiffening approach adopted by each respective program. Response-2000 adopts the tension stiffening approach proposed by Bentz [B10], whereas WHAM uses the procedure discussed previously. When tension stiffening is turned off in each respective program the moment-curvature results correlate very well to each other (refer Figure 8.15), providing good validation that the fibre element analysis engine written for WHAM works as intended and produces good results.

8.2.2 Experimental Validation

The experimental validation was performed by comparing the back-bone force-displacement response calculated using WHAM against 16 test specimens. The 16 test specimens consisted of the two cast in-situ wall specimens presented in Chapter 6 and another 14 test specimens from literature that were tested as part of four different test programs (i.e. [D3, L9, T5, T8]). The 16 test specimens had a wide range of parameters, which included shear-span ratios that varied from 2.0 to 6.5, axial load ratios varying from 0.035 to 0.128 and vertical reinforcement ratios varying from 0.005 to 0.071. A summary of the 16 test specimens is presented Table 8.1 and the cross sections of each test specimen from literature is presented in Figure 8.16. For further specimen details the reader is directed to the relevant test program cited in Table 8.1.
Figure 8.13: Stress-strain relationship for Response-2000 validation.

Figure 8.14: Theoretical validation of WHAM using Response-2000, with tension stiffening turned on. Left – rectangular wall comparison. Right – core wall comparison.

Figure 8.15: Theoretical validation of WHAM using Response-2000, with tension stiffening turned off. Left – rectangular wall comparison. Right – core wall comparison.
The analysis was performed using the confined and unconfined Mander et al. [M23] model for concrete and the reinforcement model that best matched the stress-strain properties of the reinforcement in that respective test program. For example, specimens that used coiled reinforcement, which does not have a yield plateau region, the Priestley et al. [P4] with no yield plateau was adopted, whereas specimens that only the yield stress, ultimate stress and ultimate strain were given for each specimen in the study, the simple bilinear model was adopted.

For the limited ductile test specimens, i.e. S01, S02, C3 and WSH4, the confined Mander et al. [M23] model with a nominal lateral confining pressure of 0.3 MPa was adopted. Whereas, for the remaining 12 specimens, the lateral confining pressure was calculated based on the reinforcement detailing (refer Figure 8.16) and the recommendations given by Mander et al. [M23], which are further discussed in Section 4.4.1.

The results of the force-displacement analysis using WHAM and the associated comparison to the corresponding test specimen is presented in Figures 8.17 to 8.19. The correlation between the results of WHAM and the test specimens from each respective test program (i.e. Chapter 6 or [D3, L9, T5, T8]) are discussed in the following five sub-sections and then followed by a brief summary discussing the overall accuracy and performance of WHAM.

Table 8.1: Summary of test specimens used to experimentally validate the analysis program.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reference</th>
<th>Shear-span ratio*</th>
<th>Axial load ratio†</th>
<th>Vert reinf. ratio‡</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>Chapter 6</td>
<td>6.5</td>
<td>0.065</td>
<td>0.018</td>
</tr>
<tr>
<td>S02</td>
<td>Chapter 6</td>
<td>6.5</td>
<td>0.077</td>
<td>0.014</td>
</tr>
<tr>
<td>C3</td>
<td>Lu et al. [L9]</td>
<td>6.0</td>
<td>0.035</td>
<td>0.005</td>
</tr>
<tr>
<td>C6</td>
<td>Lu et al. [L9]</td>
<td>4.0</td>
<td>0.035</td>
<td>0.005</td>
</tr>
<tr>
<td>WSH1</td>
<td>Dazio et al. [D3]</td>
<td>2.3</td>
<td>0.051</td>
<td>0.013</td>
</tr>
<tr>
<td>WSH2</td>
<td>Dazio et al. [D3]</td>
<td>2.3</td>
<td>0.057</td>
<td>0.013</td>
</tr>
<tr>
<td>WSH3</td>
<td>Dazio et al. [D3]</td>
<td>2.3</td>
<td>0.058</td>
<td>0.015</td>
</tr>
<tr>
<td>WSH4</td>
<td>Dazio et al. [D3]</td>
<td>2.3</td>
<td>0.057</td>
<td>0.015</td>
</tr>
<tr>
<td>WSH5</td>
<td>Dazio et al. [D3]</td>
<td>2.3</td>
<td>0.128</td>
<td>0.007</td>
</tr>
<tr>
<td>WSH6</td>
<td>Dazio et al. [D3]</td>
<td>2.3</td>
<td>0.108</td>
<td>0.015</td>
</tr>
<tr>
<td>A20-P10-S38</td>
<td>Tran and Wallace [T8]</td>
<td>2.0</td>
<td>0.073</td>
<td>0.032</td>
</tr>
<tr>
<td>A20-P10-S63</td>
<td>Tran and Wallace [T8]</td>
<td>2.0</td>
<td>0.073</td>
<td>0.071</td>
</tr>
<tr>
<td>RW1</td>
<td>Thomsen and Wallace [T5]</td>
<td>3.1</td>
<td>0.100</td>
<td>0.023</td>
</tr>
<tr>
<td>RW2</td>
<td>Thomsen and Wallace [T5]</td>
<td>3.1</td>
<td>0.070</td>
<td>0.023</td>
</tr>
<tr>
<td>TW1</td>
<td>Thomsen and Wallace [T5]</td>
<td>3.1</td>
<td>0.090</td>
<td>0.014–0.023</td>
</tr>
<tr>
<td>TW2</td>
<td>Thomsen and Wallace [T5]</td>
<td>3.1</td>
<td>0.075</td>
<td>0.014–0.015</td>
</tr>
</tbody>
</table>

* Shear-span ratio equals $M^*/(V^*L_w)$, which for a 1-DOF system equals $H_w/L_w$ (i.e. the aspect ratio).
† Axial load ratio is equal to the applied axial load divided by the product of the concrete strength multiplied by the gross cross-sectional area of the wall, i.e. $N^*/(f_{cm}A_g)$.
‡ The vertical reinforcement ratio for the specimens with concentrated regions of reinforcement refers to the end region (i.e. boundary element) ratio, whereas otherwise it refers to the average vertical reinforcement ratio of the entire cross-section.
Figure 8.16: Cross sections and reinforcement arrangement of test specimens from literature used to experimentally validate the analysis program [D3, L9, T5, T8].
Figure 8.17: Experimental validation of WHAM – test specimen comparison 1 of 3.
Figure 8.18: Experimental validation of WHAM – test specimen comparison 2 of 3.
Figure 8.19: Experimental validation of WHAM – test specimen comparison 3 of 3.
8.2.2.1 Chapter 6 – Test Specimens S01 and S02

The comparison to test specimens S01 and S02, the cast in-situ rectangular and building core specimens presented in Chapter 6 respectively, are presented in Figures 8.17(a) to 8.17(d). Figures 8.17(a) and 8.17(c) compare the response of the specimen (i.e. the first storey displacement, refer Figures 6.13 and 6.16), while Figures 8.17(b) and 8.17(d) compare the response of the equivalent 1-DOF specimen (i.e. the effective displacement, refer Figure 6.32).

The back-bone force-displacement response of specimen S01 determined using WHAM correlates quite well with the equivalent 1-DOF response (i.e. Figure 8.17(b)). The test specimen response (i.e. Figure 8.17(a)) also correlated quite well, however WHAM seemed to slightly under predict the yield displacement and the subsequent inelastic displacements by a small margin.

The displacement response of S02 was correlated somewhat well with the equivalent 1-DOF response (i.e. Figure 8.17(d)), however the force predicted by WHAM was significantly higher than what was observed during the test. It was discussed in Chapter 6 that during the construction of S02 poor vibration of the concrete occurred around the base of the specimen, which required some patch fixing after the formwork was stripped. It is believed this poor vibration meant that the concrete was locally weaker, with reduced bond strength capacity to the reinforcement, at the base of the wall in the maximum moment region, which then resulted in a gradual 10% loss in lateral capacity from about 1.0% to 2.3% lateral drift (refer Figure 8.20). It is believed that if good compaction of the concrete around the reinforcement occurred there would have been a 10–15% strength increase across this lateral drift region, as opposed to the strength degradation, which then would have resulted in quite a good match between WHAM and the response of S02. WHAM however, seemed to under predict the test specimen displacements (i.e. storey displacements), with good correlation not being observed (i.e. Figure 8.17(c)).

![Figure 8.20: Strength degradation in test specimen S02.](image)

The equations proposed by Priestley et al. [P4], which form the basis of WHAM (i.e. Equations 8.1 to 8.6), were developed for walls without lap splices where the inelastic curvature is solely concentrated at the base of the wall (i.e. in the typical plastic hinge region). Interestingly, despite the different curvature and inelastic strain distributions observed in S01 and S02 due to the lap splice at the base of the wall (refer Section 6.5), these equations still seem to predict the displacements fairly well.
8.2.2.2 Lu et al. [L9] – Test Specimens C3 and C6

Fairly good correlation was observed between test specimens C3 and C6 and the program, i.e. Figures 8.17(e) and 8.17(f) respectively. WHAM slightly underpredicted the maximum moment capacity of each specimen, particularly specimen C6 in the negative loading direction. The program was not able to capture the strength drop in C6 that occurred around -2% lateral drift, which was due to fracturing of the two extreme tensile face reinforcing bars. Lu et al. [L9] reported that these two bars buckled on the previous reversed positive load cycle of +1.5% lateral drift, which means the fracturing of these two bars and ensuing strength drop could have been a result of a low cycle fatigue induced fracturing of the reinforcement, as opposed to the ultimate strain (determined from monotonic tensile tests) being reached. WHAM can obviously predict the latter; however, the former scenario is outside the scope and ability of the program.

8.2.2.3 Dazio et al. [D3] – Test Specimens WSH1, WSH2, WSH3, WSH4, WSH5 and WSH6

Very good correlation was generally observed between test specimens WSH1 to WSH6 and the program, i.e. Figures 8.18(a) to 8.18(f) respectively. The behaviour of specimen WSH1 was predicted very well, including the fracturing of the vertical reinforcement that occurred between lateral drift values of 0.6% to 0.9%. WSH1 was detailed using the European equivalent of D500L reinforcement in Australia (i.e. low ductility reinforcement), which is why failure of the specimen occurred at much smaller lateral drift compared to the other five specimens, i.e. WSH2 to WSH6.

Specimens WSH3, WSH4 and WSH6 all had very good correlation between WHAM and the test data, particularly WSH4, which was limited ductile specimen. This very good correlation, in addition to the predicted responses of the other limited ductile rectangular specimens (i.e. S01 and C3), supports the proposed adopted nominal lateral confinement pressure of 0.3 MPa for limited ductile walls, which was discussed and proposed above in Section 8.1.2. This is further illustrated in Figures 8.21(a) and 8.21(b), which shows a comparison of specimens S01 and WSH4 respectively of how the predicted back-bone force-displacement response changes when the confined Mander et al. [M23] model with nominal 0.3 MPa lateral confining pressure is used for the core of the wall as opposed to the unconfined Mander et al. [M23] model.

![Figure 8.21](image-url)
The correlation between test specimens WSH2 and WSH5 and the program was not as high as specimens WSH1, WSH3, WSH4 and WSH6, however it was still fairly good. The program did not capture the strength degradation that occurred on the last positive and negative loading increments for both specimens. The program also slightly underestimated the strength of WSH2.

8.2.2.4 Tran and Wallace [T8] – Test Specimens A20-P10-S38 and A20-P10-S63

Fairly good correlation was observed between test specimens A20-P10-S38 (i.e. S38) and A20-P10-S63 (i.e. S63) and the program, i.e. Figures 8.19(a) and 8.19(b) respectively. Interestingly, the program predicted the strength of S38 quite well in the negative direction, however it slightly underpredicted it in the positive direction. The strength was then predicted fairly well in both the positive and negative directions for S63.

While the general back-bone force-displacement behaviour was predicted generally quite well for both specimens, the yield displacement was underpredicted for both specimens, however particularly specimen S63, which results in a significantly different effective stiffness between the actual response and the program.

8.2.2.5 Thomsen and Wallace [T5] – Test Specimens RW1, RW2, TW1 and TW2

Quite good correlation was observed between the rectangular test specimens RW1 and RW2 and the program, i.e. Figures 8.19(c) and 8.19(d) respectively. The yield displacement in specimen RW2 was slightly underestimated in the positive direction, however in the negative direction, the yield displacement was estimated fairly accurately. Otherwise, the force-displacement response was estimated fairly accurately for both specimens.

Test specimens TW1 and TW2 had relatively quite poor correlation compared to RW1 and RW2 (refer Figures 8.19(e) and 8.19(f) respectively). The general force-displacement response of TW2 in the positive direction (i.e. the flange in compression and the web in tension) was generally quite good, however then in the negative direction (i.e. the web in compression and the flange in tension) the strength was significantly overestimated for lateral drifts below about -2%. The strength of TW1 in the positive direction was underestimated and then overestimated in the negative direction. The poor correlation in this instance could be due to the effective flange width, which varies during the test, but in the model is considered as a constant width equal to the actual width of the flange. Further investigation is required in this instance.

8.2.2.6 Summary

Overall, very good correlation was observed between the experimental test data of the rectangular walls and the theoretical predictions of the test program. This included test specimens with a wide range of shear-span ratios, axial load ratios and vertical reinforcements ratios. This shows the program can quite confidently predict the back-bone force-displacement behaviour of rectangular walls and particularly, limited ductile rectangular walls, which are of particular importance to this research project.

A limited number of specimens were used to validate the programs ability to predict the back-bone force-displacement behaviour of non-rectangular walls. For the most part, the correlation was not as strong as the rectangular walls. Further research and development is recommended for non-rectangular walls, which may require the development and implementation of a different plastic hinge model specifically for non-rectangular walls of various cross sections. In the interim, with respect to non-rectangular wall sections, the program should be limited to performing only non-linear moment-curvature analyses.
8.3 Program Limitations and Proposed Future Improvements

This chapter has presented the development and validation of the analysis program WHAM. Due to the nature of the program and the primary objective of it being 'easy-to-use and transparent', some major limitations do exist. However, there are some limitations, which with additional work, can be easily overcome. The following is a list of WHAM program limitations and proposed future improvements:

1. **Diagonal edges**: the program can currently only handle horizontal and vertical edges. An easy future improvement would be to modify the mathematical procedure the program uses to subdivide the section to allow diagonal edges to be used.

2. **Non-uniformly distributed axial load**: the program currently applies the axial load at the centroid of the section. The program could be modified to allow non-uniform axial loads to be applied across the section. This function would be useful for building cores located along the edges of the building, which would result in the ‘inside’ wall segments of the building core taking the majority of the axial load and the ‘outside’ wall segments of the building core taking minimal axial load.

3. **Two-way bending**: the program currently subdivides the cross section into a finite number of horizontal fibres to perform the moment-curvature analysis. The program could be modified to subdivide the cross section into a finite horizontal and vertical grid of fibres, which would then allow a ‘non-horizontal’ neutral axis and the moment-curvature of the section could be solved for two-way or biaxial bending.

4. **MATLAB engine**: a MATLAB equivalent of the program could be written to allow more robust larger parameter parametric studies to be performed. While it is believed that the Microsoft Excel spreadsheet interface is ideal for using WHAM as a design program, a MATLAB interface would be preferred for undertaking parametric studies.

5. **Plane sections remain plane**: of the key assumptions the fibre element analysis procedure makes to calculate the moment-curvature response of the cross section is that ‘plane sections remain plane’. It was shown in Chapter 7 that precast buildings cores, due to the flexibility of the panel-to-panel connections, do not develop full composite action. It is being recommended that modifications be investigated to allow for jointed precast building cores, with a given connection stiffness, to be modelled by the program.

6. **Shear strength calculations**: a valuable addition to the program would be to include a shear strength design check. This would likely be fairly easy to incorporate and will likely be included in the next major revision of the program.

8.4 Parametric Study into the Displacement Behaviour of Limited Ductile RC Walls and Building Cores

WHAM was used to perform a parametric study investigating the displacement behaviour of limited ductile RC walls and building cores to develop simplified empirical models for determining the bilinear moment curvature response of a rectangular wall or building core cross section, which can then simply be converted to a bilinear force-displacement response using Equations 8.3 to 8.6.

8.4.1 Scope of the Parametric Study

The parametric study was undertaken for typical rectangular wall and box-shaped building core cross sections, as shown in Figure 8.22. Wall lengths of 2000, 3500 and 5000 mm were considered for the rectangular wall section. Similarly, for the building core section, three geometric
combinations were used, which were: 2500 mm long and 2500 mm wide; 3500 mm long and 2500 wide; and 2500 mm long and 3500 mm wide. The other parameters that varied for each section was the wall thickness, concrete grade, axial load ratio and reinforcement ratio. The various parameters and respective values used are summarised in Figures 8.23(a) and 8.23(b) for the rectangular and building core cross sections respectively.

**Figure 8.22:** Parametric study cross sections.

**Figure 8.23:** Parametric study overview.
The upper and lower bound values for each parameter were selected with reference to the findings from the reconnaissance survey of RC construction in Australia outlined in Chapter 3. Concrete grades of N40, N50 and S65 were adopted in the study, which correspond to concrete mixes with minimum characteristic 28-day cylinder compressive strength of 40, 50 and 65 MPa respectively. Mean in-situ compressive strengths of 42.8, 53.2 and 70.5 MPa for each concrete grade respectively were adopted for the study, as recommended by Foster et al. [F1] (refer Section 4.4.3). Wall thicknesses of 200, 250 and 300 mm and axial load ratios of 0%, 5%, 10% and 15% were also adopted in the study.

Four reinforcement ratios (i.e. 0.7%, 1.4%, 2.1% and 2.8%) were primarily used in the study, however to identify if the bar diameter effected the response of the cross section, three different bar sizes were used, which were N16, N20 and N24. Not all bar sizes were used for every reinforcement ratio, since for example using N16 bars to achieve a 2.8% reinforcement ratio would results in an unrealistic rebar configuration in the wall. It was decided that six reinforcement configurations (refer Figure 8.23) was sufficient to capture any potential effects that may result from different bar sizes. It should be noted that the actual reinforcement ratio used for each analysis run slightly differed to the values shown in Figure 8.23, as these values were the desired reinforcement ratios. The actual reinforcement ratio was dependent on the given bar size and was equal to the reinforcement ratio that was closest to the desired value while still having constant spaced vertical bars across the length or width of the section on each face of the wall. This was to ensure the reinforcement configurations used in the analysis generally matched industry standard detailing approaches, as identified by the reconnaissance survey presented previously in Chapter 3.

The three wall geometry configurations, three wall thickness variables, three concrete grade variables, four axial load ratio variables and six reinforcement ratio variables resulted in 648 individual analysis runs each for both the rectangular wall and building core cross section.

The Mander et al. [M23] unconfined concrete stress-strain relationship was adopted for the cover regions of the wall and the Mander et al. [M23] confined concrete stress-strain relationship with a nominal 0.3 MPa lateral confinement pressure (as discussed previously for limited ductile walls) was adopted for the core region of the walls (as shown in Figure 8.24(a)). D500N reinforcement and a simple bilinear stress-strain model using the mean material properties identified in Section 4.4.3 was adopted (as shown in Figure 8.24(b)).

![Figure 8.24](image-url): Stress-strain material models used for the parametric study.
8.4.2 Failure Criteria and Bilinear Approximation for Parametric Study

The failure criteria adopted in the parametric study was determined with reference to the recommendations provided in Section 5.5, where it is proposed for limited ductile RC walls or building cores that a maximum concrete compressive strain and reinforcement tensile strain of 0.6% and 5% respectively be adopted for the no collapse (i.e. ULS) performance of the structure. As such, failure for the walls in the parametric study was taken to be when either a compressive strain of 0.6% in the concrete or tensile strain of 5% in the reinforcement is reached, whichever occurs first.

A bilinear approximation of the non-linear moment-curvature response of each wall analysis was constructed around the yield moment (i.e. \( M_y \)), the maximum moment (i.e. \( M_u \)), the notional yield curvature (i.e. \( \phi_y \)), the yield curvature (i.e. \( \phi_y \)) and the ultimate curvature (i.e. \( \phi_u \)). The overstrength (i.e. \( \Omega \)) is then taken as the maximum moment divided by the yield moment and similarly, the curvature ductility (i.e. \( \mu_\phi \)) is taken as the ultimate curvature divided by the yield curvature. The effective moment of inertia of the cross section (i.e. \( I_{eff} \)) is taken as the slope of line from origin through the point of first yield (i.e. the notional yield curvature) divided by the elastic modulus of the concrete, i.e. \( I_{eff} = \left( \frac{M_y}{\phi_y} \right) / E_c \). Each of these parameters are further illustrated in Figure 8.25.

![Figure 8.25: Bilinear approximation of moment-curvature response.](image)

8.4.3 Results of the Parametric Study and Proposed Empirical Equations for Limited Ductile RC Walls and Building Cores

The results of the parametric study were used to create detailed and simplified empirical models for determining a bilinear moment-curvature response of both limited ductile rectangular walls and building cores. The moment capacity of the section in each model is calculated using the elastic modulus of the concrete, the effective moment of inertia of the cross section and the yield curvature. The moment capacity of an RC wall section is usually quite difficult to calculate using ‘simple hand calculations’ since walls are commonly detailed to have multiple vertical bars at varying locations across the depth of the section (i.e. wall length). This means that computer programs generally have to be heavily relied on to calculate their moment capacities. The proposed method however, allows a designer to approximately calculate the moment capacity with relative ease and without the need to rely on commercial software packages or other computer-based design aids.
The results of the parametric study and the development of each empirical model is presented in the following four sub-sections, i.e. Sections 8.4.3.1 to 8.4.3.4. This is followed by a fifth sub-section, i.e. Section 8.4.3.5, which presents a comparison of the proposed empirical bilinear models and actual moment-curvature response curves from the parametric study.

8.4.3.1 Detailed Bilinear Moment-Curvature Model for Limited Ductile Rectangular Walls

The results of the parametric study were used to develop empirical equations to calculate the effective moment of inertia, the yield curvature, ultimate curvature and overstrength. These four parameters allow a bilinear moment-curvature response curve to be determined with simplicity and ease, as proposed by Equations 8.12 to 8.18 (with reference to Figure 8.25).

\[
M_y = (E_c l_{eff}) \phi'_y \quad \ldots 8.12
\]

\[
M_u = (E_c l_{eff}) \phi_y \quad \ldots 8.13
\]

\[
\phi'_y = \frac{\phi_y}{\Omega} \quad \ldots 8.14
\]

\[
\phi_y = \frac{0.15 p_v - 2 p_v^2 + 0.0031}{L_w} \quad \text{where: } 0.005 \leq p_v \leq 0.035 \quad \ldots 8.15
\]

\[
\phi_u = \frac{a_1 p_v^3 + a_2 p_v^2 + a_3 p_v + a_4}{L_w} \quad \text{where: } 0.005 \leq p_v \leq 0.035 \quad \ldots 8.16
\]

\[
\frac{l_{eff}}{l_g} = p_v (10 - 30n) + 0.03 f_{cml} n + 0.1 \quad \ldots 8.17
\]

\[
\Omega = 9.1 n^2 - 3.6 n + 1.6 \quad \text{where: } 0.0 \leq n \leq 0.2 \quad \ldots 8.18
\]

Where:  
\( E_c \) = elastic modulus in accordance with AS 3600 [X6]  
\( = (2400^{1.5}) \times (0.043 \sqrt{f_{cml}}) \) where: \( f_{cml} \leq 40 \text{ MPa} \)  
\( = (2400^{1.5}) \times (0.024 \sqrt{f_{cml}} + 0.12) \) where: \( f_{cml} > 40 \text{ MPa} \)  
\( n \) = axial load ratio, i.e. \( N/(f_{cml}A_g) \)  
\( p_v \) = vertical reinforcement ratio, i.e. \( A_{sv}/A_g \)  
\( f_{cml} \) = mean in-situ concrete compressive strength  
\( a_1 \) to \( a_4 \) = curve fitting constants for curvature ductility factor  
\( a_1 \) = 3500 - 22000n  
\( a_2 \) = 1700n - 270  
\( a_3 \) = 5.8 - 37n  
\( a_4 \) = 0.135n - 0.004

The yield curvature for each wall was essentially just a function of the wall length and vertical reinforcement ratio (as shown in Figure 8.26). Equation 8.15 was then developed using curve fitting techniques to best represent the observed yield curvatures. Very good correlation between Equation 8.15 and yield curvature values from each respective rectangular wall in the parametric study was observed (refer Figure 8.26).
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Figure 8.26: Yield curvature results from the rectangular wall parametric study (left) and comparison to proposed empirical model (right).

The ratio of effective to gross moment of inertia for each wall was observed to be primarily dependent on the axial load ratio and the vertical reinforcement ratio of the wall, however at higher axial load ratios, the concrete compressive strength also affected the stiffness of the cross section (as shown in Figure 8.27). An empirical equation was developed (i.e. 8.17) that was a function of the axial load ratio, vertical reinforcement ratio and concrete compressive strength, with very good correlation being observed between predicted values using this equation and the results of the parametric study (refer Figure 8.27).

Figure 8.27: Ratio of effective moment of inertia results from the rectangular wall parametric study (left) and comparison to proposed empirical model (right).

The maximum moment capacity of the wall is then calculated by multiplying the yield curvature by the elastic modulus and the effective moment of inertia of the wall (i.e. Equation 8.13). This approach resulted in predicted moment capacities that correlated very well with the observed maximum moment capacities in the parametric study, as shown in Figure 8.29. The yield moment capacity can then be calculated by dividing the maximum moment capacity by the overstrength factor (i.e. $M_y = M_{ov}/\Omega$). This also results in values that correlate very well with the respective parametric study results, as shown in Figure 8.28.
The overstrength factor is determined using the developed empirical equation (i.e. 8.14), which is a function of the axial load on the wall. This equation also correlated very well with the respective overstrength values from the parametric study (refer Figure 8.31). The yield moment can alternatively be calculated by multiplying the notional yield curvature by the elastic modulus and the effective moment of inertia of the wall (i.e. Equation 8.12). The notional yield curvature is determined by dividing the yield curvature by the overstrength factor (i.e. Equation 8.14). Similar to the yield curvature, the empirical equation for the notional yield curvature resulted in very good correlation with the parametric study (refer Figure 8.31).

The results from the parametric study showed that the ultimate curvature for rectangular walls was primarily a function of the wall length, followed by the axial load ratio and the reinforcement ratio (refer Figure 8.30). An empirical equation was developed for the ultimate curvature using curve fitting techniques to best suit the data (i.e. Equation 8.16). The developed equation correlated reasonably well with the results of the parametric study (refer Figure 8.31), given the complexity and large number of variables that effect the ultimate curvature of a section.
Figure 8.30: Overstrength (left) and ultimate curvature (right) results from the rectangular wall parametric study.

Figure 8.31: Comparison of the rectangular wall parametric study results to the empirical overstrength (left), notional yield curvature (middle) and ultimate curvature (right) equations.

The accuracy of the detailed model for limited ductile rectangular walls is summarised in Table 8.2, which presents (for each parameter) the percentage of walls from the parametric study that the respective empirical model predicts the correct value within a ±15%, ±10% and ±5% range for. The table shows that the proposed model predicts the actual values quite well and reliably.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Symbol</th>
<th>Predicted values within ±15%</th>
<th>Predicted values within ±10%</th>
<th>Predicted values within ±5%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield moment</td>
<td>$M_y$</td>
<td>99.7%</td>
<td>86.4%</td>
<td>50.8%</td>
</tr>
<tr>
<td>Maximum moment</td>
<td>$M_u$</td>
<td>98.1%</td>
<td>88.3%</td>
<td>49.2%</td>
</tr>
<tr>
<td>Notional yield curvature</td>
<td>$\phi'_y$</td>
<td>100%</td>
<td>100%</td>
<td>86.7%</td>
</tr>
<tr>
<td>Yield curvature</td>
<td>$\phi_y$</td>
<td>100%</td>
<td>100%</td>
<td>84.7%</td>
</tr>
<tr>
<td>Ultimate curvature</td>
<td>$\phi_u$</td>
<td>91.0%</td>
<td>71.1%</td>
<td>37.0%</td>
</tr>
<tr>
<td>Overstrength</td>
<td>$\Omega$</td>
<td>100%</td>
<td>100%</td>
<td>95.1%</td>
</tr>
<tr>
<td>Moment of inertia ratio</td>
<td>$I_{eff}/I_g$</td>
<td>100%</td>
<td>99.7%</td>
<td>70.2%</td>
</tr>
</tbody>
</table>
8.4.3.2 Simple Bilinear Moment-Curvature Model for Limited Ductile Rectangular Walls

A simplified version of the detailed rectangular wall model presented in the previous section was developed and is presented by Equations 8.19 to 8.25 below (with reference to Figure 8.25). The simplified model was developed as a computationally simpler and quicker to operate alternative to the detailed model. The empirical equations adopted in the simplified model were developed in the same manner to which was described in the previous section for the detailed model. The simplicity and computational ease of the simplified model comes at the cost of the overall accuracy and its ability to precisely predict values that correlate with the actual values from the parametric study. The comparison of actual and predicted values for each parameter is presented in Figures 8.32 to 8.34.

\[ M_y = (E_c I_{eff}) \phi_y' \quad \ldots 8.19 \]
\[ M_u = (E_c I_{eff}) \phi_y \quad \ldots 8.20 \]
\[ \phi_y' = \frac{\phi_y}{\Omega} \quad \ldots 8.21 \]
\[ \phi_y = \frac{0.005}{L_w} \quad \ldots 8.22 \]
\[ \phi_u = \frac{0.023}{L_w} \quad \ldots 8.23 \]
\[ \frac{I_{eff}}{I_g} = n + 5p_v + 0.2 \quad \ldots 8.24 \]
\[ \Omega = 1.6 - 2.4n \quad \ldots 8.25 \]

Where:

- \( E_c \) = elastic modulus in accordance with AS 3600 [X6]
  - \( = (2400^{1.5}) \times (0.043\sqrt{f_{cmi}}) \) where: \( f_{cmi} \leq 40 \text{ MPa} \)
  - \( = (2400^{1.5}) \times (0.024\sqrt{f_{cmi}} + 0.12) \) where: \( f_{cmi} > 40 \text{ MPa} \)
- \( n \) = axial load ratio, i.e. \( \frac{N}{(f_{cmi}A_g)} \)
- \( p_v \) = vertical reinforcement ratio, i.e. \( \frac{A_{sv}}{A_g} \)
- \( f_{cmi} \) = mean in-situ concrete compressive strength

![Figure 8.32: Comparison of notional yield curvature (left), yield curvature (middle) and ultimate curvature (right) to the simplified rectangular wall model.](image-url)
**Figure 8.33:** Comparison of effective to gross moment of inertia ratio (left) and overstrength (right) to the simplified rectangular wall model.

**Figure 8.34:** Comparison of yield moment capacity (left) and maximum moment (right) to the simplified rectangular wall model.

The accuracy of the simplified model for limited ductile rectangular walls is summarised in Table 8.3, which presents (for each parameter) the percentage of walls from the parametric study that the respective empirical model predicts the correct value within ±15%, ±10% and ±5% range for. The table shows that despite the simplified of the model, most values are predicted to a reasonable well level of accuracy, given its simplified nature.

**Table 8.3:** Accuracy of simplified limited ductile rectangular wall model.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Symbol</th>
<th>Predicted values within ±15%</th>
<th>Predicted values within ±10%</th>
<th>Predicted values within ±5%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield moment</td>
<td>$M_y$</td>
<td>71.5%</td>
<td>45.5%</td>
<td>25.8%</td>
</tr>
<tr>
<td>Maximum moment</td>
<td>$M_u$</td>
<td>74.1%</td>
<td>51.5%</td>
<td>23.6%</td>
</tr>
<tr>
<td>Notional yield curvature</td>
<td>$\phi'_y$</td>
<td>84.7%</td>
<td>63.3%</td>
<td>37.0%</td>
</tr>
<tr>
<td>Yield curvature</td>
<td>$\phi_y$</td>
<td>84.0%</td>
<td>68.5%</td>
<td>38.7%</td>
</tr>
<tr>
<td>Ultimate curvature</td>
<td>$\phi_u$</td>
<td>46.8%</td>
<td>29.6%</td>
<td>21.0%</td>
</tr>
<tr>
<td>Overstrength</td>
<td>$\Omega$</td>
<td>100%</td>
<td>99.8%</td>
<td>87.7%</td>
</tr>
<tr>
<td>Moment of inertia ratio</td>
<td>$l_{eff}/l_g$</td>
<td>92.7%</td>
<td>83.2%</td>
<td>43.8%</td>
</tr>
</tbody>
</table>
8.4.3.3 Detailed Bilinear Moment-Curvature Model for Limited Ductile Building Cores

The results of the parametric study were used to develop an empirical model to predict the bilinear moment-curvature response of box-shaped building cores. The proposed model is presented by Equations 8.26 to 8.32 below. The model is presented in the same form as the previously described detailed model for rectangular walls.

\[ M_y = (E_c l_{eff}) \phi'_v \]  \qquad \ldots \ 8.26

\[ M_u = (E_c l_{eff}) \phi_y \]  \qquad \ldots \ 8.27

\[ \phi'_v = \frac{\phi_y}{\Omega} \]  \qquad \ldots \ 8.28

\[ \phi_y = \frac{0.13 p_v - 1.9 p_v^2 + 0.0024}{L_w} \]  \quad \text{where: } 0.005 \leq p_v \leq 0.035 \quad \ldots \ 8.29

\[ \phi_u = \frac{2.2 p_v - 31 p_v^2 + 0.0097}{L_w} \]  \quad \text{where: } 0.005 \leq p_v \leq 0.035 \quad \ldots \ 8.30

\[ \frac{l_{eff}}{l_g} = p_v (10 - 25n) + 0.025 f_{cni} n + 0.05 \]  \quad \ldots \ 8.31

\[ \Omega = 3.8n^2 - 1.5n + 1.33 \]  \quad \text{where: } 0.0 \leq n \leq 0.2 \quad \ldots \ 8.32

Where: \[E_c\] = elastic modulus in accordance with AS 3600 [X6]

\[ = (2400^{1.5}) \times (0.043 \sqrt{f_{cni}}) \]  \quad \text{where: } f_{cni} \leq 40 \text{ MPa}

\[ = (2400^{1.5}) \times (0.024 \sqrt{f_{cni}} + 0.12) \]  \quad \text{where: } f_{cni} > 40 \text{ MPa}

\[ n \] = axial load ratio, i.e. \(N^* / (f_{cni} A_g)\)

\[ p_v \] = vertical reinforcement ratio, i.e. \(A_{sv} / A_g\)

\[ f_{cni} \] = mean in-situ concrete compressive strength

The proposed empirical equation for the yield curvature of the section (i.e. Equation 8.29) is similar to the equation proposed for rectangular walls (i.e. Equation 8.15), which is a function of the wall length and vertical reinforcement ratio, however different coefficients were adopted, which provided a better fit with the results of the box-shaped building core parametric study (refer Figure 8.35). The overall width of the building core did not seem to affect the yield curvature of the section.

The ratio of effective to gross moment of inertia for the building core sections was observed to be primarily dependent on the axial load ratio and vertical reinforcement ratio, matching the observations of the rectangular wall sections (refer Figure 8.36). The concrete compressive strength however, affected the stiffness of the wall less so for the building core sections compared to the rectangular wall sections. The proposed equation (i.e. 8.31) is also similar to the equation proposed for rectangular walls, however different coefficients are proposed.
The overstrength factor was generally dependent on the axial load ratio of the section, similar to the rectangular wall sections, as shown in Figure 8.37. The value of the overstrength factor however, was on average slightly smaller ranging from approximately 1.1 to 1.4, whereas for the rectangular walls it ranged from approximately 1.2 to 1.6. The proposed equation (i.e. 8.32) has different coefficients to the rectangular wall equation to allow for this slightly lower and smaller range of values observed in the building core specimens. Very good correlation between the actual overstrength factor and the predicted values using the proposed equation was observed, as shown in Figure 8.38. It can also be seen in Figure 8.38 that very good correlation between the actual and predicted notional yield curvature was observed, which is calculated by dividing the yield curvature by the overstrength factor.

The ultimate curvature of the building core sections was generally governed by the tensile reinforcement strain limit, as opposed to the compressive concrete strain limit, which was generally the governing factor for the rectangular wall sections. This meant the ultimate curvature generally increased with an increase in the vertical reinforcement ratio (refer Figure 8.37), as opposed to decreasing like the rectangular wall sections (refer Figure 8.30). The tensile reinforcement strain limit was generally the governing factor in the building core sections due to the very large compressive flange, which results in a very small neutral axis depth.
The proposed equation for the ultimate curvature (i.e. 8.30) generally results in quite good correlation with the results of the building core parametric study, as shown in Figure 8.38. However, for some sections with high axial load ratios and high reinforcement, the failure mode occasionally switched from the reinforcement tensile strains governing to being controlled by compressive stress levels in the compression flange. In these sections, once the extreme fibre compressive strain reached the value corresponding to the peak concrete stress occurring predominantly in the web element, as opposed to the flange, a large reduction in strength occurred due to the significantly reduced area of concrete in the web relative to the flange. This occurred at extreme compressive fibre strain values smaller than the 0.6% compressive strain limit proposed above in Section 8.4.2, which was taken as the primary compressive failure criteria. This behaviour was assumed to be a 'local' flange failure and the curvature at this point was taken as the ultimate curvature for the section, resulting in the scattered reductions in the ultimate curvature seen at high axial load ratios and high vertical reinforcement ratios in Figure 8.37.

The yield moment and maximum moment capacities predicted using the empirical building core model were significantly less accurate than the rectangular wall model, as shown by the comparisons in Figures 8.39 and 8.40. This is due to the proposed equations for the yield curvature and the effective moment of inertia for the building core sections not correlating as well with the parametric study results. This can be seen by comparing the accuracy of these equations summarised in Table 8.4 with the respective accuracies for the rectangular walls in Table 8.2.
Figure 8.39: Comparison of predicted and actual results for the yield moment capacity of the box-shaped building core parametric study.

Figure 8.40: Comparison of predicted and actual results for the maximum moment capacity of the box-shaped building core parametric study.

Table 8.4: Accuracy of detailed limited ductile box-shaped building core model.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Symbol</th>
<th>Predicted values within ±15%</th>
<th>Predicted values within ±10%</th>
<th>Predicted values within ±5%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield moment</td>
<td>$M_y$</td>
<td>73.1%</td>
<td>56.2%</td>
<td>30.9%</td>
</tr>
<tr>
<td>Maximum moment</td>
<td>$M_u$</td>
<td>69.8%</td>
<td>50.6%</td>
<td>27.5%</td>
</tr>
<tr>
<td>Notional yield curvature</td>
<td>$\phi'_y$</td>
<td>100%</td>
<td>98.8%</td>
<td>80.7%</td>
</tr>
<tr>
<td>Yield curvature</td>
<td>$\phi_y$</td>
<td>99.4%</td>
<td>95.7%</td>
<td>70.5%</td>
</tr>
<tr>
<td>Ultimate curvature</td>
<td>$\phi_u$</td>
<td>87.0%</td>
<td>79.8%</td>
<td>51.9%</td>
</tr>
<tr>
<td>Overstrength</td>
<td>$\Omega$</td>
<td>100%</td>
<td>100%</td>
<td>88.1%</td>
</tr>
<tr>
<td>Moment of inertia ratio</td>
<td>$l_{eff}/l_g$</td>
<td>98.9%</td>
<td>88.6%</td>
<td>46.6%</td>
</tr>
</tbody>
</table>
8.4.3.4 *Simple Bilinear Moment-Curvature Model for Limited Ductile Building Cores*

A simplified version of the detailed building core model presented in the previous section was developed and is presented by Equations 8.33 to 8.39 below (with reference to Figure 8.25). The simplified model was developed as a computationally simpler and quicker to operate alternative to the detailed model. The empirical equations adopted in the simplified model were developed in the same manner to which was described in the previous section for the detailed model. The simplicity and computational ease of the simplified model comes at the cost of the overall accuracy and its ability to precisely predict values that correlate with the actual values from the parametric study. The comparison of actual and predicted values for each parameter is presented in Figures 8.41 to 8.43.

\[
M_y = (E_c l_{eff}) \phi_y' \quad \ldots 8.33
\]

\[
M_u = (E_c l_{eff}) \phi_y \quad \ldots 8.34
\]

\[
\phi_y' = \frac{\phi_y}{\Omega} \quad \ldots 8.35
\]

\[
\phi_y = \frac{0.004}{L_w} \quad \ldots 8.36
\]

\[
\phi_u = \frac{0.037}{L_w} \quad \ldots 8.37
\]

\[
\frac{l_{eff}}{l_g} = n + 10p_v + 0.05 \quad \ldots 8.38
\]

\[
\Omega = 1.3 - n \quad \ldots 8.39
\]

Where: 
- \(E_c\) = elastic modulus in accordance with AS 3600 [X6]
  - \(E_c = (2400^{1.5}) \times (0.043\sqrt{f_{cmi}})\) where: \(f_{cmi} \leq 40\) MPa
  - \(E_c = (2400^{1.5}) \times (0.024\sqrt{f_{cmi}} + 0.12)\) where: \(f_{cmi} > 40\) MPa
- \(n\) = axial load ratio, i.e. \(N^*/(f_{cmi}A_g)\)
- \(p_v\) = vertical reinforcement ratio, i.e. \(A_{sv}/A_g\)
- \(f_{cmi}\) = mean in-situ concrete compressive strength

*Figure 8.41:* Comparison of notional yield curvature (left), yield curvature (middle) and ultimate curvature (right) to the simplified box-shaped building core model.
The accuracy of the simplified model for limited ductile rectangular walls is summarised in Table 8.5, which presents (for each parameter) the percentage of walls from the parametric study that the respective empirical model predicts the correct value within a ±15%, ±10% and ±5% range for. The table shows that despite the simplified of the model, most values are predicted to a reasonable well level of accuracy, given its simplified nature.

**Table 8.5: Accuracy of simplified limited ductile box-shaped building core model.**

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Symbol</th>
<th>Predicted values within ±15%</th>
<th>Predicted values within ±10%</th>
<th>Predicted values within ±5%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield moment</td>
<td>$M_y$</td>
<td>74.8%</td>
<td>68.1%</td>
<td>43.1%</td>
</tr>
<tr>
<td>Maximum moment</td>
<td>$M_u$</td>
<td>70.8%</td>
<td>52.2%</td>
<td>28.7%</td>
</tr>
<tr>
<td>Notional yield curvature</td>
<td>$\phi'_y$</td>
<td>84.3%</td>
<td>76.7%</td>
<td>44.3%</td>
</tr>
<tr>
<td>Yield curvature</td>
<td>$\phi_y$</td>
<td>79.5%</td>
<td>58.2%</td>
<td>31.0%</td>
</tr>
<tr>
<td>Ultimate curvature</td>
<td>$\phi_u$</td>
<td>52.6%</td>
<td>36.3%</td>
<td>18.8%</td>
</tr>
<tr>
<td>Overstrength</td>
<td>$\Omega$</td>
<td>100%</td>
<td>100%</td>
<td>72.8%</td>
</tr>
<tr>
<td>Moment of inertia ratio</td>
<td>$I_{eff}/I_g$</td>
<td>94.4%</td>
<td>85.0%</td>
<td>54.9%</td>
</tr>
</tbody>
</table>
8.4.3.5 Comparison of Proposed Empirical Bilinear Models and Concluding Remarks

The proposed bilinear empirical models for rectangular and box-shaped building cores are compared against six respective moment-curvature results from the parametric study in Figures 8.44 and 8.45 respectively. The parameters of each cross section used for the comparisons in Figures 8.44 and 8.45 are summarised in Tables 8.6 and 8.7 respectively. While visually comparing the proposed models against the results of only 6 out of 648 cross sections within each parametric study may not be completely representative of how well the models correlate overall with the results, Figures 8.44 and 8.45 show very good correlation between the proposed models and the actual non-linear moment-curvature response curves from the study. The various figures and tables in the previous sub-section for each respective model show that the proposed models are able to predict the moment-curvature response of both rectangular and box-shaped building core cross sections quite well with a reasonably high level of accuracy, especially given the simplicity of the models and the large range of variables and unknowns that effect the non-linear behaviour of RC walls.

Table 8.6: Summary of parameters for rectangular wall model comparisons in Figure 8.44.

<table>
<thead>
<tr>
<th>(a) cross section 1</th>
<th>(b) cross section 2</th>
<th>(c) cross section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_w = 3500$ mm</td>
<td>$L_w = 5000$ mm</td>
<td>$L_w = 5000$ mm</td>
</tr>
<tr>
<td>$t_w = 250$ mm</td>
<td>$t_w = 250$ mm</td>
<td>$t_w = 250$ mm</td>
</tr>
<tr>
<td>$f_{cmi} = 53.2$ MPa</td>
<td>$f_{cmi} = 53.2$ MPa</td>
<td>$f_{cmi} = 53.2$ MPa</td>
</tr>
<tr>
<td>$n = 0.05$</td>
<td>$n = 0.05$</td>
<td>$n = 0.05$</td>
</tr>
<tr>
<td>$p_v = 0.0144 (d_b = 20$ mm)</td>
<td>$p_v = 0.0146 (d_b = 20$ mm)</td>
<td>$p_v = 0.0211 (d_b = 20$ mm)</td>
</tr>
</tbody>
</table>

Table 8.7: Summary of parameters for building core model comparisons in Figure 8.45.

<table>
<thead>
<tr>
<th>(a) cross section 1</th>
<th>(b) cross section 2</th>
<th>(c) cross section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_w = 2500$ mm</td>
<td>$L_w = 2500$ mm</td>
<td>$L_w = 3500$ mm</td>
</tr>
<tr>
<td>$b_w = 2500$ mm</td>
<td>$b_w = 2500$ mm</td>
<td>$b_w = 2500$ mm</td>
</tr>
<tr>
<td>$t_w = 250$ mm</td>
<td>$t_w = 250$ mm</td>
<td>$t_w = 250$ mm</td>
</tr>
<tr>
<td>$f_{cmi} = 53.2$ MPa</td>
<td>$f_{cmi} = 53.2$ MPa</td>
<td>$f_{cmi} = 53.2$ MPa</td>
</tr>
<tr>
<td>$n = 0.05$</td>
<td>$n = 0.05$</td>
<td>$n = 0.05$</td>
</tr>
<tr>
<td>$p_v = 0.0149 (d_b = 20$ mm)</td>
<td>$p_v = 0.0298 (d_b = 24$ mm)</td>
<td>$p_v = 0.0219 (d_b = 20$ mm)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(d) cross section 4</th>
<th>(e) cross section 5</th>
<th>(f) cross section 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_w = 2500$ mm</td>
<td>$L_w = 2500$ mm</td>
<td>$L_w = 3500$ mm</td>
</tr>
<tr>
<td>$b_w = 2500$ mm</td>
<td>$b_w = 2500$ mm</td>
<td>$b_w = 2500$ mm</td>
</tr>
<tr>
<td>$t_w = 250$ mm</td>
<td>$t_w = 250$ mm</td>
<td>$t_w = 250$ mm</td>
</tr>
<tr>
<td>$f_{cmi} = 53.2$ MPa</td>
<td>$f_{cmi} = 53.2$ MPa</td>
<td>$f_{cmi} = 53.2$ MPa</td>
</tr>
<tr>
<td>$n = 0.10$</td>
<td>$n = 0.10$</td>
<td>$n = 0.10$</td>
</tr>
<tr>
<td>$p_v = 0.0144 (d_b = 20$ mm)</td>
<td>$p_v = 0.0298 (d_b = 24$ mm)</td>
<td>$p_v = 0.0219 (d_b = 20$ mm)</td>
</tr>
</tbody>
</table>
Figure 8.44: Moment-curvature comparison of the rectangular wall model.

Figure 8.45: Moment-curvature comparison of the box-shaped building core model.
8.4.4 Force Reduction Factors for Limited Ductile RC Walls and Building Cores

The results of the parametric study for the rectangular wall and building core cross sections were used to perform a second parametric study to assess how the force reduction factor \( R_f = \Omega \mu \) of these wall sections are affected by the various wall parameters in the initially parametric study (refer Figure 8.23). The force reduction factor was taken as the ultimate displacement divided by the displacement associated with the point of first yield (i.e. notional yield displacement), as represented by Equation 8.40. The force reduction factor also equals the overstrength multiplied by the displacement ductility (i.e. \( R_f = \Omega \mu \)), where the overstrength is equal to the yield displacement divided by the notional yield displacement (i.e. \( \Omega = \Delta_y / \Delta_y' \)) and the displacement ductility is equal to the ultimate displacement divided by the yield displacement (i.e. \( \mu = \Delta_u / \Delta_y \)), which then results in \( R_f = \Delta_u / \Delta_y' \) (i.e. Equation 8.40). The ultimate displacement was calculated using the Priestley et al. [P4] plastic hinge model, which was the same model adopted in WHAM and described previously.

\[
R_f = \frac{\Delta_u}{\Delta_y'} \quad ... \text{8.40}
\]

Where:
- \( \Delta_y' \) = point of first yield (i.e. Equation 8.3)
- \( \Delta_u \) = ultimate displacement (i.e. Equation 8.5)

The parametric study into the behaviour of the force reduction factor was performed for both rectangular RC walls and box-shaped building core cross sections. The validation of the Priestley et al. [P4] plastic hinge model using WHAM (i.e. Section 8.2.2) showed very good and reliable correlation for the rectangular wall sections. Therefore, the results of this parametric study for the rectangular walls could be considered quite robust and indicative of their true behaviour. However, given the less reliable correlation for the non-rectangular wall cross sections observed in the validation process in Section 8.2.2, the results of this parametric study for the building core sections should be considered as indicative only.

The force reduction factor for each wall cross section was calculated using a range of effective height (i.e. \( H_{eff} \)) values, which correlated to shear-span ratios varying from 3.0 to 8.0. The walls were assumed to be cantilevered elements with a single point load at the top of the wall (i.e. a 1-DOF system), which means the shear-span ratio is equal to the \( H_{eff} / L_w \). It was deemed more appropriate to compare the force reduction factor for different length walls for equal shear-span ratios, rather than effective heights, since the latter could result in misleading observations.

The results of the parametric study into the force reduction factor of limited ductile rectangular walls and box-shaped building cores are presented in Figures 8.46 and 8.47 respectively. It was generally observed that for the majority of rectangular walls the force reduction factor was calculated to be less than 2.6, which is the recommended value for ‘limited ductile’ RC construction in the Australian earthquake standard, AS 1170.4 [X4]. However, the majority of the building core sections had values greater than 2.6, but still less than 4.5, which is the recommended value for ‘moderately ductile’ RC construction in AS 1170.4. The failure mechanism for the majority of the rectangular walls were controlled by concrete compressive strains (i.e. compression-face controlled, refer Section 4.3.4.1), which resulted in lower ultimate curvatures and the resulting lower force reduction factors. The majority of the building core sections however, were controlled by reinforcement tensile strains (i.e. tension-face controlled, refer Section 4.3.4.1), which resulted in larger ultimate curvatures and the resulting larger force reduction factors.
However, as discussed in Chapters 6 and 7, it should be noted that comparing the force reduction factors from AS 1170.4 directly to these results can be somewhat misleading since the force reduction factor in this instance is calculated using the actual yield displacement of the wall. Whereas when force-based analysis is performed using AS 1170.4 the yield displacement of the building or structure is essentially controlled by the empirical equation used for calculating the first mode natural period (i.e. Equation 2.7, refer Section 2.4.2), which is a very conservative equation. Therefore, AS 1170.4 underestimates the yield displacement (as a result of the conservative natural period calculation), which means higher force reduction factors would be required to factor up the smaller yield displacement to achieve the equivalent ultimate displacement. These results however, do provide caution against blindly adopting the higher ‘moderately’ ductile classification in AS 3600 without adopting the much more onerous detailing requirements recommended in Chapter 3 (i.e. Table 3.4), particularly for rectangular walls.

The force reduction factors, for the of shear-span ratios considered, ranged from 1.6 to 5.1 for the rectangular walls and 2.0 and 5.6 for the building cores, however the majority of values ranged from 1.7 to 3.6 and 2.1 to 5.3, as shown in Figures 8.46 and 8.47 respectively. It was observed that the wall length (for the same shear-span ratios), wall thickness and the concrete compressive strength had a minimal effect on the force reduction factor, as shown in Figures 8.46(a) to 8.46(d) and 8.47(a) to 8.47(d). The shear-span ratio affected the force reduction factor significantly more than the wall length or concrete strength, where the factor increased between 15–30% and 25–45% when the shear-span ratio decreased from 8.0 to 3.0 for rectangular and building core sections respectively. The shear-span ratio had a larger effect on the building core sections since they generally exhibited a tension-face controlled flexure failure mechanism, which meant larger ultimate curvatures were developed for the equivalent depth (i.e. wall length) section. Since the yield displacement is largely controlled by the yield curvature, which is in turn, is primarily controlled by the wall length (meaning the yield displacement is generally a constant for a given wall length regardless of the failure mechanism), the tension-face controlled failure mechanisms allowed larger force reduction factors to be developed in the building core sections.

The parameter that appeared to have the largest effect on the force reduction factor for the rectangular wall sections was the axial load ratio, as shown in Figures 8.46(e) and 8.46(f). Further, the force reduction factor increased on average by approximately 50% when the axial load ratio was decreased from 15% to 5% (i.e. Figure 8.46(a) vs. 8.46(b) or Figure 8.46(c) vs. 8.46(d)). The percentage of vertical reinforcement also had a somewhat significant effect on the force reduction factor, however this was only observed for lower axial load ratios (e.g. 5% in Figures 8.46(g) and 8.46(h)). When the axial load ratio was increased to 15%, the percentage of vertical reinforcement had only a marginal effect on the force reduction factor, as shown in Figures 8.46(g) and 8.46(h).

The axial load ratio generally had a lesser effect on the force reduction factor for the building core specimens, with the percentage of vertical reinforcement or the shear-span ratio being the dominant parameter that affected its value (refer Figure 8.47). Interestingly though, when the axial load ratio and the percentage of vertical reinforcement both increased significantly the factor reduced substantially (refer Figures 8.47(f), 8.47(g) and 8.47(h)). In these few circumstances (e.g. $p_v = 0.022$ & $n = 0.15$ or $p_v = 0.030$ & $n = 0.15$) the failure mechanism switched from being tension-face controlled to compression-face controlled. This means, broadly speaking, the axial load ratio is the dominant parameter that governs the amount of inelastic behaviour in compression-face controlled wall sections and the shear-span ratio (followed second by the percentage of vertical reinforcement) is the dominant parameter that governs the amount of inelastic behaviour in tension-face controlled wall sections.
$f_{cmi} = 52.3 \text{ MPa} \quad n = 0.05 \quad \rho_v \sim 0.015 \quad (d_w = 20 \text{ mm})$

(a) variables: wall length and thickness

$\rho \approx 0.05 \quad (d_w = 20 \text{ mm})$

(b) variables: wall length and thickness

$\rho \approx 0.15 \quad (d_w = 20 \text{ mm})$

(c) variables: concrete grade and wall length

$\rho \approx 0.05 \quad (d_w = 20 \text{ mm})$

(d) variables: concrete grade and wall length

$\rho \approx 0.15 \quad (d_w = 20 \text{ mm})$

(e) variables: axial load ratio and reinf. ratio

$\rho \approx 0.05 \quad (d_w = 20 \text{ mm})$

(f) variables: axial load ratio and reinf. ratio

$\rho \approx 0.15 \quad (d_w = 20 \text{ mm})$

(g) variables: reinf. ratio and axial load ratio

$\rho \approx 0.05 \quad (d_w = 20 \text{ mm})$

(h) variables: reinf. ratio and axial load ratio

$\rho \approx 0.15 \quad (d_w = 20 \text{ mm})$

Figure 8.46: force reduction factors for limited ductile RC rectangular walls.
$f_{cmi} = 52.3 \text{ MPa} \quad n = 0.05 \quad p_{v~0.015} (d_b = 20 \text{ mm})$

(a) variables: wall length and thickness

$t_w = 250 \text{ mm} \quad n = 0.05 \quad p_{v~0.015} (d_b = 20 \text{ mm})$

(c) variables: concrete grade and wall length

$\sigma = 250 \text{ mm} \quad n = 0.05 \quad p_{v~0.015} (d_b = 20 \text{ mm})$

(d) variables: concrete grade and wall length

$\sigma = 250 \text{ mm} \quad n = 0.15 \quad p_{v~0.015} (d_b = 20 \text{ mm})$

(e) variables: axial load ratio and reinf. ratio

$\sigma = 250 \text{ mm} \quad n = 0.15 \quad p_{v~0.015} (d_b = 20 \text{ mm})$

(f) variables: axial load ratio and reinf. ratio

$\sigma = 3500 \text{ mm} \quad n = 0.15 \quad p_{v~0.015} (d_b = 20 \text{ mm})$

(g) variables: reinf. ratio and axial load ratio

$\sigma = 3500 \text{ mm} \quad n = 0.15 \quad p_{v~0.015} (d_b = 20 \text{ mm})$

(h) variables: reinf. ratio and axial load ratio

Figure 8.47: force reduction factors for limited ductile RC box-shaped building cores.
8.5 Conclusions

This chapter has outlined the development of a simple, user-friendly and transparent analysis program for predicting the back-bone force-displacement behaviour of slender (i.e. flexure-controlled) RC walls and building cores. The program is called WHAM and is written using Microsoft Excel spreadsheets and will be released as a free-of-charge design tool for practicing structural engineers or as an educational tool for undergraduate students or researchers. The program was validated using both theoretical and experimental approaches. The theoretical validation of WHAM was conducted by performing moment-curvature analyses on walls with different cross sections and axial load ratios and comparing the results against a widely used commercial software package called RAPT and a sectional analysis program developed by researchers at the University of Toronto called Response-2000. Very good correlation was observed between the results from WHAM and the two independent software packages, providing strong validation that the fibre element analysis engine written for WHAM works as intended and produces good results.

The experimental validation was performed by comparing the back-bone force-displacement response calculated using WHAM against the results of the two cast in-situ test specimens presented in Chapter 6 (i.e. test specimens S01 and S02) and another 14 test specimens from literature that were tested as part of four different test programs. Very good correlation was observed between WHAM and the rectangular wall test specimens, which included a wide variation of walls with shear-span ratios ranging from 2.0 to 6.5, axial load ratios from 3.5% to 12.8% and percentages of vertical reinforcement from 0.5% to 7.1%. Only a limited number of comparisons were performed against non-rectangular wall test specimens, however the correlation was not as good for these sections.

The program was used to perform a parametric study into limited ductile rectangular walls and building cores. The results of the parametric study were used to develop detailed and simplified empirical models for both rectangular walls and building cores, which can be used to quickly determine a bilinear moment-curvature response without performing any complex calculations or needing to use computer-based design aids. The parametric study was also used to study how the force reduction factor (i.e. \( R_f = \Omega \mu \) or \( R_f = \Delta u / \Delta p \)) in limited ductile rectangular walls and building cores was affected for different wall lengths, wall thickness, concrete grades, axial load ratios, vertical reinforcement ratios and vertical reinforcement bar diameter. It was shown that the shear-span ratio, axial load ratio and percentage of vertical reinforcement all significantly affect the value of the factor (i.e. this shows that each of these parameters significantly affects the amount of inelastic displacement a wall can develop).

The force reduction factor decreases for increasing shear-span ratios, percentages of vertical reinforcement and axial load ratios. Rectangular walls were typically compression-face controlled, which meant the concrete compressive strains governed the ultimate displacement of the section, whereas the building core sections were typically tension-face controlled, which meant the reinforcement tension strains govern the ultimate displacement of the section. The axial load ratio was the dominant factor affecting the amount of inelastic behaviour in compression-face controlled walls, where the force reduction factor was increased by approximately 50% as the axial load ratio decreased from 15% to 5%. Whereas for tension-face controlled walls, decreasing the shear-span ratio had the largest effect on increasing the force reduction factor, followed by decreasing the percentage of vertical reinforcement and decreasing the axial load ratio.
Chapter 9

Conclusions and Recommendations

9.1 Summary and Conclusions

Based on the conclusions presented at the end of each chapter, the following major findings from the research project have been observed:

Part A – Design Methodology and RC Wall Design and Construction in Australia

A1. Seismic design in Australia is based around achieving a life-safety performance objective under a relatively short return period earthquake event with no consideration given to capacity design principles in any Australian codes or standards. This has left the Australian building stock vulnerable and susceptible to partial or complete structural collapses in the event of a very rare long return period earthquake.

A2. Using current force-based analysis procedures, designing buildings to resist the much larger earthquake actions associated with very rare long return period events would result in significant impact to industry. It would likely result in wholesale changes such as larger building cores, additional or thicker walls with increased reinforcement, which would invariably encounter push back by building professionals. The nature of force-based ULS design is such that it will inadvertently always under predict the true ULS capacity of a building. Higher levels of seismic performance could, in many scenarios, likely be attained in RC wall buildings simply through the adoption of better detailing practices and establishing a rationale flexure based (i.e. ductile) yielding mechanism in the major RC wall elements. Meaning these wholesale changes mentioned above would not be required. However, it is very difficult to accurately predict performance using traditional force-based ULS techniques. Using more rational displacement-based seismic design and assessment techniques, buildings could more easily, quickly and accurately be assessed to determine compliance. The development of a displacement-based seismic design standard and its implementation in practice requires knowledge of the force-displacement behaviour of both the overall building and individual lateral load resisting elements.
A3. The majority of RC walls in Australia are either individual isolated rectangular walls or box-shaped buildings cores that surround emergency exit stairwells or lift shafts. Cast in-situ walls and building cores are typically detailed with a continuous mat of vertical and horizontal reinforcement on each face of the wall, with uniform bar spacing across the length of the wall in both directions. The reinforcement detailing around wall openings, wall intersections and end regions of walls generally consisted of additional ‘U’ bars matching the main reinforcement bar size and spacing. Industry standard practice also includes the use of lap splices of the vertical reinforcement at the base of the wall in the plastic hinge region. Confinement is rarely specified in the end regions of RC walls. RC wall detailing in Australia would broadly be defined as limited ductile in the vast majority of structures.

A4. In recent years, the adoption of precast concrete walls and jointed precast building cores over traditional cast in-situ RC elements have become increasingly popular in low and mid-rise construction. In conjunction with this rapid rise in popularity has also come the adoption of some detailing practices that are inconsistent with the commonly used ductility assumptions in (the currently adopted) force-based seismic design procedure. These include detailing RC walls with low ductility reinforcement, central layers of vertical reinforcement or low percentages of vertical reinforcement (i.e. less than about 0.7%). It is recommended that a new ductility classification, namely ‘non-ductile’, be adopted when these detailing practices are used, which has a displacement ductility of 1.0 and an overstrength factor of 1.3, resulting in a force reduction factor of 1.3.

A5. Force-based seismic analysis assumes strength is being traded for ductility through the use of the force reduction factor, which is a combination of a displacement ductility factor and structural performance (or overstrength) factor. Consideration needs to be given as how the structure can develop the amount ductility being assumed and what affect this will have on other structural elements in the building. Designers should adopt the ‘limited ductile’ structure classification to AS 1170.4 [X4] and AS 3600 [X6], which allows for a displacement ductility and overstrength factor of 2.0 and 1.3 to be used respectively. The ‘moderately ductile’ classification should only be adopted when the more onerous detailing requirements recommended in Chapter 3 are adopted. The moderately ductile classification has a much higher force reduction factor of 4.5, compared to the limited ductile factor that is equal to 2.6. The 73% higher force reduction factor does not come for free and requires the associated higher level of detailing to ensure the extra ductility is achieved.

A6. The overall system behaviour of a building must be considered. Precast walls around the perimeter of a building (often referred to as load bearing only elements), cannot be assumed to take zero lateral load by assigning 100% of the lateral load to the central core. While it may appear that these perimeter walls take minimal lateral load when distributing elastic forces based on initial or effective stiffnesses, the real forces could be somewhat different and larger when the overall system’s inelastic displacement is taken into consideration. When performing seismic analysis, which trades strength for ductility, displacement compatibility of the whole system needs to be thoroughly evaluated and assessed.

A7. There is little research or guidance available in literature for assessing the displacement behaviour of limited ductile RC walls. The majority of guidance provided in literature is for ductile RC walls, which are generally detailed to have well confined end regions and no lap splices of the vertical reinforcement in the plastic hinge region.
A8. Mean material strengths for standard grades of reinforcement and concrete in Australia are recommended. The recommended reinforcement properties were made using a large database of tensile test data that included 285 rebar tensile tests performed at Swinburne University of Technology and another 6,257 rebar tensile tests performed by an independent material testing laboratory, whom are contracted by industry suppliers to assess code compliance of their reinforcing bars. The database included test data from a 7-year period from 2011 to 2017.

A9. A significant amount of experimental testing programs into the lateral displacement behaviour of RC walls have been performed in the last 15 years, particularly following the 2010 Chile and 2011 Christchurch earthquakes, where many examples of poor performance in RC walls were observed. Fortunately, very rarely did these instances result in complete structural collapse of the building. The majority of these testing programs are focused at the lateral performance of ductile RC walls and hence not directly relevant to Australian construction practices.

Part B – Experimental Testing of Limited Ductile RC Walls and Components

B1. The boundary element prism testing showed that RC elements subject to cyclic axial tension-compression loading (i.e. the type of loading the end regions of RC walls are essentially subjected to) undergo a bifurcation effect, which is dependent on the amount of inelastic plastic strain the vertical reinforcement is subjected to during a respective tension displacement cycle. After load reversal when the element is subjected to axial compression, the element will either ‘overcome’ the residual plastic tension strains and regain its initial axial stiffness or the residual plastic tension strains will initiate a global out-of-plane buckling or local bar buckling failure mechanism, which results in axial load failure of the element.

B2. The boundary element prism testing suggested that RC walls detailed with two layers of vertical reinforcement (i.e. one per face) can sustain local tensions of about 6–7% before bar buckling occurs in the subsequent reversed load cycle.

B3. Out-of-plane buckling modes were also observed in the boundary element prism testing; however, further research is still required before a comprehensive model can be proposed to accurately predict the tension strains required to initial the failure mode in the subsequent reversed load cycle. In the interim, the Paulay and Priestley [P11] model, which was shown to be somewhat conservative, could be adopted. It was also shown that strain rate seems to have little effect on the out-of-plane buckling mechanism, which confirmed that the pseudo-static loading protocols used for the test specimens were appropriate for experimentally assessing this behaviour.

B4. The singularly reinforced prism specimens behaved poorly in comparison to the specimens detailed with two layers of vertical bars (i.e. one per face). The singularly reinforced specimens were subject to local bar buckling at lower local tension strain values in previous reversed load cycles and also had larger out-of-plane movement relative to the ‘doubly’ reinforced specimens.

B5. The results of the boundary element prism testing were used to develop and propose a set of reinforcement and concrete strain limits, which could be used in displacement-based assessment procedures of limited ductile RC walls in Australia or other regions of lower seismicity that predominantly utilise limited ductile RC construction.
B6. A series of cyclic tension-compression tests on individual rebar samples were performed. These tests showed that cyclic loading does not affect (i.e. reduce) the ultimate stress or strain of the bar, unless the bar buckles in the previous compression cycle. Where minor buckling occurred in the compression cycles, the ultimate strain (i.e. uniform elongation) was reduced to about 60% of the monotonic value.

B7. The boundary element prism testing was used to develop and validate a tension stiffening model for limited ductile RC walls. Very good correlation was observed between the results of the prism testing and the proposed tension stiffening model. The model was used to development a global average strain to local reinforcement strain relationship for limited ductile RC elements.

B8. The cast in-situ rectangular wall and building core test specimens achieved a displacement ductility of about 2 before significant strength degradation initiated, which is in good agreement with the ductility assumptions usually adopted by Australian designers when using the Australian earthquake loading standard, AS 1170.4. The ultimate failure mechanism of the rectangular wall was crushing of the concrete in the extreme compressive fibre of the wall, whereas the building core specimen failed due to the development of high tensile strains in the vertical reinforcement, which resulted in a combination of fracturing of the vertical reinforcement, unzipping of the lap splice and degrading of the concrete due to bond failure between the concrete and reinforcement.

B9. The rectangular wall and building core specimens were able to achieve ±3.1% and ±2.3% lateral drift respectively prior to lateral load failure of the specimens occurring (i.e. the lateral strength dropped below 80% of the respective maximum capacities). The walls continued to achieve ±4.2% and ±4.5% lateral drift respectively prior to axial load failure occurring (i.e. complete structural collapse).

B10. The test results of both cast in-situ test specimens showed that a traditional plastic hinge with distributed cracking and distributed plasticity, as commonly seen in RC wall testing, was not achieved due to the lap splice at the base of the wall. The lap splice created a region of overstrength, over which only hairline cracks formed; major cracks formed at the base of the wall and the top of the lap splice. The plastic rotation and curvature of the wall was concentrated at these two locations.

B11. The jointed precast concrete building core specimens, despite the lateral strength decreasing significantly (often below 20% of the maximum response), could withstand very large in-plane lateral drifts prior to axial load failure occurring (i.e. complete structural collapse). Test specimens S03, S04 and S05 were loaded to 6.5%, 8.0% and 4.9% drift respectively without axial load failure occurring, after which the test was terminated.

B12. The welded stitch plate (WSP) connections in the system level tests were not stiff enough to allow full composite action to be developed. This resulted in the effective moment of inertia of test specimen S05 being 25% lower than the equivalent cast in-situ building core specimen (i.e. test specimen S02). Further, it also meant that only 80% of the theoretical maximum moment capacity of the wall was developed before flexure failure occurred via fracturing of the vertical reinforcement.

B13. The precast building core specimen that was detailed using low ductility reinforcement was only able to develop a marginal amount of ductility before fracturing of the vertical reinforcement occurred, which then resulted in a significant and sudden reduction in lateral strength.
B14. Poor grouting was observed at the base of one of the precast building core specimens, which resulted in a stress concentration and premature compression failure due to vertical tensile splitting of the compression flange panel. This would likely be mitigated by providing nominal cross ties at the top and bottom of the panel, which would mean the panel would not solely be relying on the concrete's tensile strength to prevent this type of local failure mechanism. It is being recommended that all precast panels be detailed with a minimum of one nominal cross-tie at each grout tube connection.

B15. The vertical shear force transfer between the adjacent panels to allow composite action to be developed in the section was not evenly distributed among the WSP connections. The shear forces were concentrated towards the connections at the base of the wall.

B16. The precast building core specimens that had dowel bar areas less than the cross-sectional area of the vertical reinforcement in the panel above developed one large crack at the base of the wall with all the inelastic plastic tensile strain concentrated at this location. This allowed a rocking mechanism to develop, which then allowed for very large in-plane lateral drifts without axial load failure. However, this may not be a desired yielding mechanism as it would likely provide little strength or resistance to torsional actions, which are common design actions for building cores in multi-storey buildings.

B17. The first of the two new innovative prototype connections, i.e. the ‘grouted panel pocket’ (GPP) connection, was 1.73 times stronger and 1.73 times stiffer than the baseline WSP connection. The test provided proof-of-concept that the GPP connection is potentially a viable substitute for WSP connections in jointed precast building cores. However, more testing would be required to establish comprehensive design rules for this type of connection to allow widespread industry adoption. It is likely the strength of the GPP connection could be substantially increased by providing confinement reinforcement in the grouted pocket, either in the form of 6 mm diameter helical ligatures or fibre reinforced grout.

B18. The second of the two new innovative prototype connections, i.e. the ‘post tensioned corbel’ (PTC) connection had a much greater strength and stiffness than the WSP and GPP connections. The stiffness of the PTC connection was greater than 40 times that of the WSP and GPP connections. The test provides proof-of-concept that the PTC is potentially a viable substitute for wet joints in jointed precast building cores. The PTC connection appears to be a very effective method for constructing jointed precast building cores. Its design would be based on well-established structural engineering principles and could be adopted in industry immediately.

B19. A blind prediction study was performed with multiple structural engineering design consultancies in Australia to assess the strength of a WSP connection. The study showed that there is widespread opinion on how to calculate the capacity of these connections, with predictions varying by almost a factor of five.

B20. A series of recommended details for WSP, GPP and PTC connections are presented in Chapter 7. Ultimate vertical shear strengths and vertical stiffness of each connection is also provided.
Part C – Modelling the Displacement Behaviour of Limited Ductile RC Walls

C1. Chapter 8 outlined the development of a simple, user-friendly and transparent analysis program for predicting the back-bone force-displacement behaviour of slender (i.e. flexure-controlled) RC walls and building cores. The program is called WHAM and it is written using Microsoft Excel spreadsheets and will be released as a free-of-charge design tool.

C2. WHAM was validated using the cast in-situ test specimens presented in Chapter 6 (i.e. test specimens S01 and S02) and another 14 test specimens from literature that were tested as part of four different test programs. Very good correlation was observed between WHAM and the rectangular wall test specimens, which included a wide variation of walls with shear-span ratios ranging from 2.0 to 6.5, axial load ratios from 3.5% to 12.8% and percentages of vertical reinforcement from 0.5% to 7.1%. Only a limited number of comparisons were performed against non-rectangular wall test specimens; however, the correlation was not as good for these sections.

C3. WHAM was used to perform a parametric study into limited ductile rectangular walls and building cores. The results of the parametric study were used to develop detailed and simplified empirical models for both rectangular walls and building cores, which can be used to quickly determine a bilinear moment-curvature response without performing any complex calculations or needing to use computer-based design aids.

C4. The parametric study was also used to study how the amount of inelastic displacement varied in limited ductile rectangular walls and building cores for different wall lengths, wall thicknesses, concrete grades, axial load ratios, vertical reinforcement ratios, vertical reinforcement bar diameter and shear-span ratios. The study showed that rectangular walls were generally compression-face controlled, which means the concrete compressive strains governed the maximum displacement capacity of the section, whereas the building core sections were generally tension-face controlled, which means the reinforcement tensile strains governed the maximum displacement capacity of the section. The axial load ratio was the dominant factor affecting the amount of inelastic behaviour in the compression-face controlled walls, where the force reduction factor was increased by approximately 50% as the axial load ratio decreased from 15% to 5%. Whereas for tension-face controlled walls, decreasing the shear-span ratio had the largest effect on increasing the force reduction factor, followed by decreasing the percentage of vertical reinforcement and decreasing the axial load ratio.

9.2 Recommendations for Further Research

This research project has studied the inelastic displacement behaviour of RC walls and building cores in Australia. Based on the conclusions and findings of this research project, the following recommendations for further research are being made:

1. This research project has primarily concentrated on the inelastic displacement behaviour of individual RC walls. Further research is recommended into the overall system behaviour of limited ductile RC wall buildings. Numerous case study building examples were identified during the reconnaissance survey presented in Chapter 3, which would be ideal case studies for undertaking further numerical modelling and analysis to assess how the displacement behaviour of RC walls and building cores is affected by the overall system response of the structure. Key areas of interests would be wall-floor interaction and torsional behaviour.
2. Further experimental programs into the displacement behaviour of limited ductile RC walls and buildings cores is recommended to assess and better understand (i) the bi-directional loading response and (ii) the combined bi-directional and torsional loading response. This testing could be performed using traditional quasi-static loading protocols or by undertaking hybrid analysis using the MAST System at Swinburne University of Technology.

3. Analytical work is recommended to assess and better understand the vertical shear force distribution amongst stitch plate connections in jointed precast building cores. This work would ideally be undertaken by performing non-linear time history analyses of both low and mid-rise case study buildings. The analysis should be performed on both individual isolated jointed precast building core models and system level models, which would include multiple building cores and floor slabs. The floor slabs, depending how they're constructed, would also provide coupling between adjacent panels in jointed building cores. The separate individual and system level models would be used to assess how much contribution the floor slabs provide to the overall composite action of the core. Another primarily outcome of this modelling work would be to provide recommendations or guidance to what the minimum vertical connection stiffness requirements are to ensure the individual panels can response together as one equivalent composite section.

4. Finite element model of limited ductile RC building cores is recommended to better understand the inelastic displacement behaviour. This modelling work could also be used to assess how the torsional response of precast cores is affected when one large crack is developed at the base of the section with all the inelastic tension strains concentrated at this one location.

5. This research project has developed and undertaken proof-of-concept experimental testing of an innovative new prototype connection for precast building cores, namely the 'grouted panel pocket' (GPP) connection. Further testing of the GPP connection is recommended to allow a robust design model for the connection to be developed. The additional testing is recommended to be performed on thicker 200 or 250 mm panels using larger M24 ferrules. Test specimens with various forms of confinement to the grouted pocket is recommended.

6. Finally, a research project is recommended to further develop and enhance the sectional analysis program WHAM, which is presented in Chapter 8. A number of proposed future improvements and recommendations to WHAM are outlined in Section 8.3 in detail and briefly summarised below:
   i. Re-write the mathematical procedure for subdividing the section to allow cross sections with diagonal edges to be used.
   ii. Provide an additional function in the program that allows non-uniformly distributed axial loads to be applied to the cross section.
   iii. Write an alternative version of the program that can subdivide the cross section both horizontally and vertically to allow a 'diagonal neutral axis' and two-way bending of the section.
   iv. Write a MATLAB version of the program to allow faster and more robust parametric studies to be performed.
   v. Provide an additional function in the program to allow precast building cores to be modelled with a degree of flexibility between adjacent panels.
   vi. Provide an additional function in the program that assesses the shear strength of the section.
References


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Appendix A

Test Specimen Drawings

Appendix A contains the detailed construction drawings used to construct the boundary element prism test specimens (i.e., test specimens P01 to P17), the large-scale RC wall test specimens (i.e., test specimens S01 to S05) and the precast panel connection specimens (i.e., test specimens J01 to J03). Table A.1 summarises all the construction drawings which were produced for manufacturing (i) the concrete test specimens and (ii) other associated components required for testing (e.g., structural steel loading brackets). The construction drawings of each test specimen are highlighted in Table A.1 in bold typeface for clarity, with a page number reference to each test specimens associated drawing/s. The construction drawings of the other associated components required for testing are noted in Table A.1 for completeness only and have not been included within this Appendix.

Table A.1: Summary of construction drawings.

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**Drawing 1:** Test specimen P01 and P02 (Chapter 5).

*Drawing not printed to scale.*
Appendix A: Test Specimen Drawings

Drawing 2: Test specimen P03 and P04 (Chapter 5)

TYPICAL WALL SECTION
1:10 AT A3

TOP BOUNDARY ELEMENT SECTION
1:10 AT A3

GENERAL NOTES
G1 CONCRETE COVER IN WALL SECTION TO BE AS PER WALL SECTION DETAIL.
G2 CONCRETE COVER IN BOUNDARY ELEMENTS TO BE 20mm.
G3 MAX AGGREGATE SIZE TO BE 20mm.
G4 CONCRETE BUMP TO BE 80mm.
G5 ALL REINFORCEMENT TO BE GRADE D500M TO AS/NZS 4671:2001.
G6 CURE CONCRETE SPECIMENS.
G7 CPBW DENOTES COMPLETE PénéTRATION BUTT WELD.
G8 ALL WELDING OF REINFORCEMENT TO BE IN ACCORDANCE WITH AS 1554.3.2000 Part 5.

DRAWING ISSUES AND AMENDMENTS
VERSION DATE DESCRIPTION
A 17/10/14 INITIAL DRAWING ISSUE

PROJECT TITLE:
CYCLIC AXIAL STRENGTH OF CONCRETE WALLS

CONCRETE TEST SPECIMEN TYPE 1

DRAWER
S J MENEGON

DRAWN BY
S J MENEGON

DRAWING NUMBER
005 / 002.20A

DATE
17/10/2014
Drawing 3: Test specimen P05 and P06 (Chapter 5).

GENERAL NOTES
G1 CONCRETE COVER IN WALL SECTION TO BE AS PER WALL SECTION DETAIL.
G2 CONCRETE COVER IN BOUNDARY ELEMENTS TO BE 20mm.
G3 MAX AGGREGATE SIZE TO BE 20mm.
G4 CONCRETE SUPPLY TO BE 80mm.
G5 ALL REINFORCEMENT TO BE GRADE D550 TO AS/NZS 4871:2001.
G6 CURE CONCRETE SPECIMENS.
G7 CPBW DENOTES COMPLETE PENETRATION BUTT WELD.
G8 ALL WELDING OF REINFORCEMENT TO BE IN ACCORDANCE WITH AS 1864.3:2009 Part 3.

CROSS SECTION
WALL SPECIMEN DETAILS
1:20 AT A3

SIDE ELEVATION

TOP BOUNDARY ELEMENT SECTION
1:10 AT A3

BOTTOM BOUNDARY ELEMENT SECTION
1:10 AT A3
Drawing 4: Test specimen P07 (Chapter 5).

Drawing not printed to scale.
Drawing 5: Test specimen P08 (Chapter 5).
**Drawing 6:** Test specimen P09 (Chapter 5).

Drawing not printed to scale.
**Drawing 8**: Test specimen P12 (Chapter 5).

Drawing not printed to scale.
Drawing 9: Test specimen P14 (Chapter 5).

Drawing not printed to scale.
**Drawing 10:** Test specimen P11 (Chapter 5).

Drawing not printed to scale.
Drawing 11: Test specimen P13 (Chapter 5).

Drawing not printed to scale.
**Drawing 12:** Test specimen P15 (Chapter 5).

Drawing not printed to scale.
**Drawing 13:** Test specimen P16 (Chapter 5).
**Drawing 14:** Test specimen P17 (Chapter 5).

Drawing not printed to scale.
Drawing 15: Test specimen S01 (Chapter 6).

Drawing not printed to scale.
**Drawing 16:** Test specimen S02 (Chapter 6) – drawing 1 of 3.

*Drawing not printed to scale.*
**Drawing 17:** Test specimen S02 (Chapter 6) – drawing 2 of 3.

*Drawing not printed to scale.*
Drawing 18: Test specimen S02 (Chapter 6) – drawing 3 of 3.

Drawing not printed to scale.
Drawing 20: Test specimen S03 (Chapter 7) – drawing 2 of 2.
Drawing 21: Test specimen S04 (Chapter 7) – drawing 1 of 2.

Drawing not printed to scale.
Drawing 22: Test specimen S04 (Chapter 7) – drawing 2 of 2.

Drawing not printed to scale.
Drawing 23: Test specimen S05 (Chapter 7) – drawing 1 of 5.

Drawing not printed to scale.
**Drawing 24:** Test specimen S05 (Chapter 7) – drawing 2 of 5.

Drawing not printed to scale.
**Drawing 25:** Test specimen S05 (Chapter 7) – drawing 3 of 5.

*Drawing not printed to scale.*
Drawing 26: Test specimen S05 (Chapter 7) – drawing 4 of 5.

Drawing not printed to scale.
**Drawing 28:** Test specimen J01 (Chapter 7) – drawing 1 of 2.

Drawing not printed to scale.
Drawing 29: Test specimen J01 (Chapter 7) – drawing 2 of 2.

Drawing not printed to scale.
**Drawing 30:** Test specimen J02 (Chapter 7) – drawing 1 of 2.

**Concrete Notes:**
- C1: Concrete to be grade 850 (pre-cast concrete, way mark).
- C2: 20mm clear cover to reinforcement.
- C3: Cure concrete panel.

**Drawing Details:**
- Type: 17.0F bar
- Section B-B: Plan as cast legend
- Section C-C: Reinforcement
- Section D: Precast panel connection testing
- Section A-A: Plan as cast

**Plan as Cast Legend:**
- Denotes 3.0m wide red edge lift
- Denotes finish panel with doube steel reinforcemnet

**General Notes:**
- G1: If concrete surface elements with a tolerance of 3mm or less
- G2: Panel weight is 56.0 metric tonnes
- G3: Log reinforcement elements with a tolerance of 3mm or less

**Note:** Drawing not printed to scale.
Drawing 31: Test specimen J02 (Chapter 7) – drawing 2 of 2.

Drawing not printed to scale.
**Drawing 32:** Test specimen J03 (Chapter 7) – drawing 1 of 2.

Drawing not printed to scale.
Drawing 33: Test specimen J03 (Chapter 7) – drawing 2 of 2.
Appendix B

Supplement to Chapter 5 – Boundary Element Testing

Appendix B is a supplement Chapter 5 and provides additional material and test data associated with the first major component of experimental work documented therein. This appendix provides additional test results for specimens P02 to P04 and P06 to P17 in the form of data summary tables, which were used to construct the backbone tension response curves used to validate the tension stiffening model developed, also in Chapter 5. The stress strain curves used in the tension stiffening model to validate against the test specimens are also presented within.

B.1 Supplementary Test Data

The tension force-displacement response curves for each test specimen, which were used to validate the tension stiffening model, are summarised in Tables B.1 and B.2 for test specimens P02 to P10 (excluding P05) and P11 to P17 respectively.

B.2 Stress Strain Curves

The stress strain curves used to model the tension displacement behaviour of the test specimens are presented in Figures B.1 to B.4. The stress strain curves for test specimens P02 to P04 (i.e. Figure B.1) were constructed using Priestley, Calvi and Kowalsky [P4] stress strain curve, the yield stress of the reinforcement and mean response values for D500N reinforcement that were established in Chapter 4. Similarly, the stress strain curves for test specimens P07 to P10 (i.e. Figure B.2) were constructed by 'matching' the Priestley et al. [P4] stress strain curve to the strain values determined using the post-yield strain gauges and the corresponding average stress in the reinforcement at that time. This indirect approach for test specimens P02 to P04 and P07 to P10 was adopted since no reinforcement material samples were taken for these test specimens.
The stress strain curves for test specimens P11 to P17 (i.e. Figures B.3 and B.4) however, were calculated using reinforcement material samples from each respective test specimen. A reinforcement sample was taken from each individual bar in each test specimen prior to constructing them. The average stress strain reinforcement curve for each test specimen was calculated by taking the average of the individual curves for each individual bar. This can be seen in Figures B.3 and B.4 where the individual stress strain curves are shown in light grey and the average response is shown as a darker thicker solid line.

Table B.1: Test specimens P02 to P04 and P06 to P08 tension increment backbone data.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Quantity</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
<th>Cycle 7</th>
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<tbody>
<tr>
<td>P02</td>
<td>Force (kN)</td>
<td>204.7</td>
<td>310.2</td>
<td>416.4</td>
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<td>Stress (MPa)</td>
<td>301.9</td>
<td>457.5</td>
<td>614.1</td>
<td>627.6</td>
<td>641.3</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Disp. (mm)</td>
<td>2.6</td>
<td>4.7</td>
<td>19.5</td>
<td>39.6</td>
<td>59.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P03</td>
<td>Force (kN)</td>
<td>376.1</td>
<td>565.1</td>
<td>751.3</td>
<td>774.9</td>
<td>809.9</td>
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<td>Stress (MPa)</td>
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<td>623.0</td>
<td>642.5</td>
<td>671.6</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>Disp. (mm)</td>
<td>3.5</td>
<td>5.5</td>
<td>16.8</td>
<td>36.9</td>
<td>56.5</td>
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<td>Force (kN)</td>
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<td>540.1</td>
<td>737.8</td>
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<td>5.1</td>
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* Strain denotes local strain of the reinforcement measured using post-yield strain gauges.
Table B.2: Test specimens P09 to P17 tension increment backbone data.

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<td>Stress (MPa)</td>
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<td>696.9</td>
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<td>Disp. (mm)</td>
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<td>16.9</td>
<td>25.1</td>
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<td>Strain* (%)</td>
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<td>636.5</td>
<td>646.5</td>
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<td>657.6</td>
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<td>Stress (MPa)</td>
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<td>Force (kN)</td>
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* Strain denotes local strain of the reinforcement measured using post-yield strain gauges.
Figure B.1: Stress-strain curves (test specimens P02 to P04).

Test specimen P02
N12 grade D500N

Test specimens P03 and P04
N16 grade D500N

Figure B.2: Stress-strain curves (test specimens P07 to P10).

Test specimen P07
N10 grade D500N

Test specimen P08
N16 grade D500N

Test specimen P09
N10 grade D500N

Test specimen P10
N16 grade D500N
Figure B.3: Stress-strain curves (test specimens P11 to P15).
Figure B.4: Stress-strain curves (test specimens P16 and P17).
Appendix C

Supplement to Chapter 6 – Cast In-Situ RC Wall Testing

Appendix C is a supplement to Chapter 6 and provides additional material and background to the second major component of experimental work documented therein. This appendix provides additional test results for specimens S01 and S02, including: tabulated test data to construct backbone curves; rotation, flexure, shear and total displacement profiles; crack progression maps; and damage progression photos. Following this, test data corrections which were used to account for slip and rotations within the test setup in the MAST System have been documented and included. Derivations for the deflection and stiffness of cantilever elements, which have been referred to in Chapter 6, are documented and included in this appendix also. The appendix is concluded with a discussion into the verification methods which were used to assess the accuracy of the photogrammetry system and to ensure the MAST System maintained the moment-controlled behaviour for the duration of the test.

C.1 Supplementary Test Data

C.1.1 Displacement Profiles

Displacement profiles (including flexure, shear, shear sliding and total), rotation profiles and profiles showing the percentage contribution of each type of deformation, for increasing lateral drift increments, are presented in Figures C.1 and C.2 and Figures C.3 and C.4 and for test specimens S01 and S02 respectively.

C.1.2 Backbone Curves

Peak data from each cycle of the test, which can be used to construct backbone curves of the specimens response, has been tabulated in Tables C.1 and C.2 for test specimen S01 and Tables C.3 and C.4 for test specimen S02 for the convenience of the reader.
Figure C.1: Displacement profiles for test specimen S01 – negative direction loading.
Figure C.2: Displacement profiles for test specimen S01 – positive direction loading.
Figure C.3: Displacement profiles for test specimen S02 – negative direction loading.
Figure C.4: Displacement profiles for test specimen S02 – positive direction loading.
### Table C.1: Backbone data, test specimen S01, cycle 1.

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### Table C.4: Backbone data, test specimen S02, cycle 2.

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C.2 Crack Mapping and Damage Progression

C.2.1 Crack Mapping

Crack progression was mapped and recorded on the second positive and negative loading cycle for both specimens. The progression of cracking in test specimen S01 for both positive and negative loading directions is presented in Figure C.7. The progression of cracking in test specimen S01 for the positive and negative loading directions is presented in Figures C.8 and C.9 respectively. For test specimen S02 crack mapping is shown for the rear face of the specimen (i.e. the web section) and the tension flange for the respective loading direction being considered. A key plan showing the tension flange for each loading direction is provided in Figure C.5.

![Figure C.5: Key plan for crack progression figures.](image)

C.2.2 Damage Progression

Damage progression photos are presented in Figures C.10 and C.11 for the test specimen S01 and Figures C.12 and C.13 for test specimen S02. For test specimen S02 the damage progression photos show the rear face of the specimen (i.e. the web section). A key plan showing where the damage progression photos were taken from is provided in Figure C.6.

![Figure C.6: Key plan for damage progression figures.](image)
Figure C.7: Crack mapping of test specimen S01 – positive and negative loading directions.
Figure C.8: Crack mapping of test specimen S02 – positive loading directions.
Figure C.9: Crack mapping of test specimen S02 – negative loading directions.
Note: refer to Figure C.6 for key plan showing the location the photo series is taken from.

**Figure C.10:** Damage progression for test specimen S01 – positive loading direction.
Note: refer to Figure C.6 for key plan showing the location the photo series is taken from.

Figure C.11: Damage progression for test specimen S01 – negative loading direction.
(a) cycle 55  
+0.5% drift  

(b) cycle 65  
+0.8% drift  

(c) cycle 75  
+1.1% drift  

(d) cycle 85  
+1.5% drift  

(e) cycle 95  
+2.3% drift  

(f) cycle 105  
+3.4% drift  

Note: refer to Figure C.6 for key plan showing the location the photo series is taken from.

**Figure C.12:** Damage progression for test specimen S02 – positive loading direction.
(a) cycle 57  
-0.5% drift

(b) cycle 67  
-0.7% drift

(c) cycle 77  
-1.1% drift

(d) cycle 87  
-1.5% drift

(e) cycle 97  
-2.2% drift

(f) cycle 107  
-3.3% drift

Note: refer to Figure C.6 for key plan showing the location the photo series is taken from.

**Figure C.13:** Damage progression for test specimen S02 – negative loading direction.
C.3 Test Data Correction for Rigid Body Rotation and Base Sliding

The measured test data was corrected to account for rigid body rotation and base sliding, as shown in Figures C.14(a) and C.14(b) respectively. Rigid body rotation of the specimen occurs when there is insufficient post tensioning force between the bottom boundary element and the strong floor, allowing uplift on the tension side to occur when lateral load is applied. Similarly, base sliding occurs when there is insufficient friction between the base of the bottom boundary element and the strong floor, which is largely dependent on the level of post tensioning provided.

![Diagram of rigid body rotation and base sliding](image)

**Figure C.14**: Test data correction for rigid body rotations and base sliding.

The in-plane displacement due to rigid body rotation and base sliding needs to be subtracted from the measured in-plane displacement so the 'true' displacement of the specimen is measured and reported, i.e. Equation C.1. The displacement due to rigid body rotation and base sliding for each test specimen was determined using Equations C.2 and C.3 respectively. The left and right horizontal and vertical displacements (i.e. $\Delta_h$ and $\Delta_v$), as shown in Figure C.14, were measured using the laser displacement sensors (refer Chapter 6).

$$
\Delta_{x,corrected} = \Delta_{x,measured} - (\Delta_{rocking} + \Delta_{sliding}) 
$$  \hspace{1cm} \text{... C.1}

$$
\Delta_{rocking} = \left( \frac{\Delta_{h,right} - \Delta_{h,left}}{L} \right) H_w 
$$  \hspace{1cm} \text{... C.2}

$$
\Delta_{sliding} = 0.5(\Delta_{h,right} + \Delta_{h,left}) 
$$  \hspace{1cm} \text{... C.3}
### C.4 Instrumentation Summary

A summary of physical instrumentation used during the testing of test specimens S01 and S02 is presented in Table C.5.

**Table C.5: Summary of physical instrumentation – test specimens S01 and S02.**

<table>
<thead>
<tr>
<th>Instrumentation</th>
<th>Description</th>
<th>Stroke</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>L01-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 1 right</td>
</tr>
<tr>
<td>L02-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 1 left</td>
</tr>
<tr>
<td>L03-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 2 right</td>
</tr>
<tr>
<td>L04-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 2 left</td>
</tr>
<tr>
<td>L05-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 3 right</td>
</tr>
<tr>
<td>L06-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>30 mm</td>
<td>LVDT stack row 3 left</td>
</tr>
<tr>
<td>L07-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>30 mm</td>
<td>LVDT stack row 4 right</td>
</tr>
<tr>
<td>L08-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>30 mm</td>
<td>LVDT stack row 4 left</td>
</tr>
<tr>
<td>L09-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>20 mm</td>
<td>LVDT stack row 5 right</td>
</tr>
<tr>
<td>L10-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>30 mm</td>
<td>LVDT stack row 5 left</td>
</tr>
<tr>
<td>L11-SPOT</td>
<td>String potentiometer</td>
<td>600 mm</td>
<td>Horizontal displacement 2</td>
</tr>
<tr>
<td>L12-SPOT</td>
<td>String potentiometer</td>
<td>600 mm</td>
<td>Horizontal displacement 1</td>
</tr>
<tr>
<td>L13-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Vertical displacement 1</td>
</tr>
<tr>
<td>L14-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Vertical displacement 2</td>
</tr>
<tr>
<td>L15-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Vertical displacement 3</td>
</tr>
<tr>
<td>L16-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Vertical displacement 4</td>
</tr>
<tr>
<td>L17-LASER</td>
<td>Laser displacement sensor</td>
<td>100 mm</td>
<td>MAST sliding</td>
</tr>
<tr>
<td>L18-LASER</td>
<td>Laser displacement sensor</td>
<td>100 mm</td>
<td>Foundation block sliding</td>
</tr>
<tr>
<td>L19-LASER</td>
<td>Laser displacement sensor</td>
<td>100 mm</td>
<td>MAST sliding</td>
</tr>
<tr>
<td>L20-LASER</td>
<td>Laser displacement sensor</td>
<td>100 mm</td>
<td>Foundation block sliding</td>
</tr>
<tr>
<td>L21-LASER</td>
<td>Laser displacement sensor</td>
<td>200 mm</td>
<td>MAST rotation</td>
</tr>
<tr>
<td>L22-LASER</td>
<td>Laser displacement sensor</td>
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<td>L23-LASER</td>
<td>Laser displacement sensor</td>
<td>200 mm</td>
<td>MAST rotation</td>
</tr>
<tr>
<td>L24-LASER</td>
<td>Laser displacement sensor</td>
<td>200 mm</td>
<td>Foundation block rotation</td>
</tr>
</tbody>
</table>
C.5 Photogrammetry Verification

Verification of the photogrammetry system (i.e. the V-STS N series by Geodetic Systems) was performed by comparing the in-plane lateral displacement of test specimens S01 and S02 that was recorded using the physical instrumentation against the displacement values determined using the photogrammetry system, i.e. Figures C.15 and C.16 respectively. A comparison was also performed for the top rotation of specimen S01, i.e. Figure C.17. It can be seen in these figures that very good correlation between the photogrammetry system and the physical instrumentation was observed.

![Figure C.15: Photogrammetry verification – lateral x-axis displacement – test specimen S01.](image)

![Figure C.16: Photogrammetry verification – lateral x-axis displacement – test specimen S02.](image)

![Figure C.17: Photogrammetry verification – top y-axis rotation – test specimen S02.](image)
C.6 MAST System Loading Protocol Verification

C.6.1 General

The MAST System is state-of-the-art test machine capable of applying six degree-of-freedom (DOF) loading to a test specimen. The testing required the MAST System to run a mixed-mode loading protocol; requiring two DOFs to be controlled in force mode and the remaining four DOFs controlled in displacement mode. The loading protocol is summarised below in Table C.6.

Table C.6: The MAST System loading protocol summary.

<table>
<thead>
<tr>
<th>Degree of freedom</th>
<th>Mode</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_x$ – x-axis translation</td>
<td>Displacement</td>
<td>Refer Chapter 6</td>
</tr>
<tr>
<td>$T_y$ – y-axis translation</td>
<td>Displacement</td>
<td>Zero movement</td>
</tr>
<tr>
<td>$T_z$ – z-axis translation</td>
<td>Force</td>
<td>Constant force *</td>
</tr>
<tr>
<td>$R_x$ – x-axis rotation</td>
<td>Displacement</td>
<td>Zero rotation</td>
</tr>
<tr>
<td>$R_y$ – y-axis rotation</td>
<td>Force</td>
<td>$M_y = kF_x$ †</td>
</tr>
<tr>
<td>$R_z$ – z-axis rotation</td>
<td>Displacement</td>
<td>Zero rotation</td>
</tr>
</tbody>
</table>

* The axial force for test specimen S01 and S02 was -585kN and -1200kN respectively.
† The specimens were tested with a shear-span ratio of 6.5, which resulted in a $k$ factor of 5.2 (as discussed in Chapter 6).

For the duration of the test the MAST System was successfully programmed to maintain a zero displacement in the out-of-plane $y$-axis, zero rotation about both the $x$-axis and $z$-axis and the desired axial force in the vertical $z$-axis. This has been illustrated in Figures C.18 and C.19.

C.6.2 Moment Controlled Behaviour

The testing required a $y$-axis moment to be applied continuously, which was a coupled to the $x$-axis force. The $y$-axis moment was continuously being re-calculated based on the live $x$-axis force at intervals equal to the system clock rate, which was 1024 Hertz. There was approximately a 2-millisecond delay between the $y$-axis moment calculation and application to the specimen.

The ratio of the $y$-axis moment divided by the $x$-axis force was plotted with respect to time to investigate whether the MAST System successfully applied the moment-controlled behaviour for the duration of the test, as presented in Figures C.20 and C.21 for test specimens S01 and S02 respectively. On initial inspective it appears as though the moment-controlled behaviour was not successfully applied, i.e. Figures C.20(a) and C.21(a). However, the noise seen here occurred when the specimens were loaded back towards zero displacement in the $x$-axis, which means the associated $x$-axis force was equal to zero or a very small number approaching zero. When the signal is corrected to remove the data corresponding to when the specimens are crossing the origin point (i.e. zero $x$-axis displacement), the $y$-axis moment to $x$-axis force ratio becomes approximately constant at the desired value of 5.2, as shown in Figures C.20(b) and C.21(b). It should be noted that the noise seen towards the end of the corrected signal for specimen S01 in Figure C.20(b) corresponds to the complete loss of lateral strength that was observed towards the end of the testing regime (refer Chapter 6).
Figure C.18: Individual degree-of-freedom response – test specimen S01.

Figure C.19: Individual degree-of-freedom response – test specimen S02.
Figure C.20: Moment controlled behaviour – test specimen S01.

Figure C.21: Moment controlled behaviour – test specimen S02.
Appendix D

Supplement to Chapter 7 – Precast RC Wall and Connection Testing

Appendix D is a supplement to Chapter 7 and provides additional material and background to the third major component of experimental work documented therein. The appendix consists of seven sub-sections as follows: Section D.1 provides supplementary test data for specimens S03 to S05; Section D.2 provides crack progression maps for specimens S03 to S05 and damage progression photos for specimen S05; Section D.3 provides time lapse photos showing the construction sequence and test setup of specimen S05; Section D.4 provides a summary table of the physical instrumentation used during the testing of specimens S03 to S05; Section D.5 provides damage progression photos for specimens J01 to J03; Section D.6 provides background design theory for welded stitch plate connections; and Section D.7 provides a summary table of the physical instrumentation used during the testing of specimens J01 to J03.

D.1 Precast Building Core Testing: Supplementary Test Data

D.1.1 Displacement Profiles

Displacement profiles for test specimen S05 (including flexure, shear, shear sliding and total), rotation profiles and profiles showing the percentage contribution of each type of deformation, for increasing lateral drift increments are presented in Figures D.1 and D.2 for negative and positive loading directions respectively. Profiles for test specimen S03 and S04 are not provided.

D.1.2 Backbone Curves

Peak data from each cycle of the test, which can be used to construct backbone curves of the specimens response, has been tabulated in Tables D.1 and D.2 for test specimen S03, Tables D.3 and D.4 for test specimen S04 and Tables D.5 and D.6 for test specimen S05 for the convenience of the reader.
Figure D.1: Displacement profiles for test specimen S05 – negative direction loading.
Figure D.2: Displacement profiles for test specimen S05 – positive direction loading.
Table D.1: Backbone data, test specimen S03, cycle 1.

<table>
<thead>
<tr>
<th>Ref.</th>
<th>$\Delta x$</th>
<th>$\gamma_x$</th>
<th>$F_x$</th>
<th>$F_x/F_{max}$</th>
<th>Ref.</th>
<th>$\Delta x$</th>
<th>$\gamma_x$</th>
<th>$F_x$</th>
<th>$F_x/F_{max}$</th>
</tr>
</thead>
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<td></td>
</tr>
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<td>-0.1</td>
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<td>-0.1</td>
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<td>-0.2</td>
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</tr>
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<td>0.4</td>
<td>170.7</td>
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<td>-0.3</td>
<td>-189.0</td>
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<td>-0.5</td>
<td>-221.4</td>
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<td>-0.7</td>
<td>-244.2</td>
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</tr>
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<td>0.93</td>
<td>73</td>
<td>-27.9</td>
<td>-1.1</td>
<td>-251.7</td>
<td>1.00</td>
</tr>
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<td>1.6</td>
<td>292.2</td>
<td>0.97</td>
<td>83</td>
<td>-37.5</td>
<td>-1.4</td>
<td>-252.6</td>
<td>1.00</td>
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Table D.2: Backbone data, test specimen S03, cycle 2.

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<tr>
<th>Ref.</th>
<th>$\Delta x$</th>
<th>$\gamma_x$</th>
<th>$F_x$</th>
<th>$F_x/F_{max}$</th>
<th>Ref.</th>
<th>$\Delta x$</th>
<th>$\gamma_x$</th>
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<td>%</td>
<td>kN</td>
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<td>%</td>
<td>kN</td>
<td></td>
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Table D.3: Backbone data, test specimen S04, cycle 1.

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<th>( \gamma_x ) %</th>
<th>( F_x ) kN</th>
<th>( \frac{F_x}{F_{\text{max}}} )</th>
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<th>( \gamma_x ) %</th>
<th>( F_x ) kN</th>
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<td>-1.8</td>
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<td>1.00</td>
<td>103</td>
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</tr>
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<td>-4.6</td>
<td>-112.6</td>
<td>0.37</td>
</tr>
</tbody>
</table>

Table D.4: Backbone data, test specimen S04, cycle 2.

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Table D.5: Backbone data, test specimen S05, cycle 1.

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Table D.6: Backbone data, test specimen S05, cycle 2.

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D.1.3 Stitch Plate Loads

The vertical shear forces in the eight-welded stitch plate (WSP) connections for test specimen S05 were determined using the backbone force-displacement behaviour of the matching WSP connection component level test (i.e. test specimen J01), as discussed in Chapter 7. This section presents additional data not presented in Chapter 7. The vertical shear force and displacement of each WSP connection in test specimen S05 for load cycles 15, 17, 35, 37, 55, 57, 75 and 77 are summarised in Tables D.7 and D.8. The stitch plate reference labels are presented in Figure D.3.

![specimen S05 cross section](image)

**Figure D.3:** Stitch plate reference plan and elevation.
### Table D.7: Test specimen S05 stitch plate forces – front face.

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<tr>
<th>Load cycle</th>
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<th>Stitch plate #1</th>
<th>Stitch plate #1</th>
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<td>Disp. (mm)</td>
<td>Force (kN)</td>
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<td></td>
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<td>-0.6% drift</td>
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<td></td>
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<tr>
<td>Load cycle 77</td>
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### Table D.8: Test specimen S05 stitch plate forces – rear face.

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<td>Disp. (mm)</td>
<td>Force (kN)</td>
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<td>Load cycle 55</td>
<td>0.03</td>
<td>0.4</td>
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<td>0.01</td>
<td>0.1</td>
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<td>-0.6% drift</td>
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D.2 Precast Building Core Testing: Crack Mapping and Damage Progression

D.2.1 Crack Mapping
Crack progression was mapped and recorded on the second positive and negative loading cycle for all three specimens. The progression of cracking in test specimens S03, S04 and S05 for the positive and negative loading directions is presented in Figures D.6 and D.7, Figures D.8 and D.9 and Figures D.10 and D.11 respectively. The crack mapping for each specimen is shown for the rear face of the specimen (i.e. the web section) and the tension flange for the respective loading direction being considered. A key plan showing the tension flange for each loading direction is provided in Figure D.4.

![Figure D.4: Key plan for crack progression figures.](image)

D.2.2 Damage Progression
Damage progression photos are presented in Figures D.12 and D.13 for the test specimen S05. Additionally, damage progression photos for the two critical stitch plates (i.e. stitch plate #1 and #2) are presented in Figures D.14 and D.15. A key plan showing where the damage progression photos were taken from is provided in Figure D.5.

![Figure D.5: Key plan for damage progression figures.](image)
Figure D.6: Crack mapping of test specimen S03 – positive loading directions.
Figure D.7: Crack mapping of test specimen S03 – positive loading directions.
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<tr>
<td>65</td>
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<td>95</td>
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</table>

Figure D.8: Crack mapping of test specimen S04 – positive loading directions.
Figure D.9: Crack mapping of test specimen S04 – positive loading directions.
Figure D.10: Crack mapping of test specimen S05 – positive loading directions.
Figure D.11: Crack mapping of test specimen S05 – positive loading directions.
(a) cycle 45
+0.4% drift

(b) cycle 55
+0.6% drift

(c) cycle 65
+0.8% drift

(d) cycle 75
+1.1% drift

(e) cycle 85
+1.6% drift

(f) cycle 95
+2.3% drift

Note: refer to Figure D.5 for key plan showing the location the photo series is taken from.

**Figure D.12**: Damage progression for test specimen S05 – photo location #1 – positive loading.
Appendix D: Supplement to Chapter 7 – Precast RC Wall and Connection Testing

Figure D.13: Damage progression for test specimen S05 – photo location #1 – negative loading.

Note: refer to Figure D.5 for key plan showing the location the photo series is taken from.
(a) cycle 35, +0.3% drift

(b) cycle 37, -0.3% drift

(c) cycle 45, +0.4% drift

(d) cycle 47, -0.4% drift

(e) cycle 55, +0.6% drift

(f) cycle 57, -0.6% drift

(g) cycle 65, +0.8% drift

(h) cycle 67, -0.9% drift

Figure D.14: Welded stitch plate movement in test specimen S05 (1 of 2).
D.3 Precast Building Core Testing: Manufacturing and Assembly

The RC precast panels for test specimen S05 were manufactured by a local precast company in Melbourne and transported to the Smart Structures Laboratory at Swinburne where they were erected and assembled to form the jointed precast building core specimen. Time lapse photos showing the manufacturing of the panels, erection and assembly, welding and grouting and specimen preparations prior to testing are shown in Figures D.16, D.17, D.18 and D.19 respectively.

Figure D.15: Welded stitch plate movement in test specimen S05 (2 of 2).
Figure D.16: Test specimen S05 construction process, 1 of 4 – panel manufacturing.
(a) panels ready for assembly in the SSL  
(b) panel number 1 erected  

(c) panel number 2 erected  
(d) panel number 3 erected  

(e) panel number 4 erected  
(f) top boundary element placement  

**Figure D.17:** Test specimen S05 construction process, 2 of 4 – panel assembly.
Figure D.18: Test specimen S05 construction process, 3 of 4 – welding and grouting.
Figure D.19: Test specimen S05 construction process, 4 of 4 – instrumentation and setup.
### D.4 Precast Building Core Testing: Instrumentation Summary

A summary of physical instrumentation for test specimen S05 is presented in Table D.9. Refer to Table C.5 for a summary of the physical instrumentation used for test specimens S03 and S04.

<table>
<thead>
<tr>
<th>Instrumentation</th>
<th>Description</th>
<th>Stroke</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>L01-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 1 right</td>
</tr>
<tr>
<td>L02-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 1 left</td>
</tr>
<tr>
<td>L03-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>20 mm</td>
<td>LVDT stack row 2 right</td>
</tr>
<tr>
<td>L04-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>30 mm</td>
<td>LVDT stack row 2 left</td>
</tr>
<tr>
<td>L05-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>20 mm</td>
<td>LVDT stack row 3 right</td>
</tr>
<tr>
<td>L06-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>30 mm</td>
<td>LVDT stack row 3 left</td>
</tr>
<tr>
<td>L07-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 4 right</td>
</tr>
<tr>
<td>L08-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 4 left</td>
</tr>
<tr>
<td>L09-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>50 mm</td>
<td>LVDT stack row 5 right</td>
</tr>
<tr>
<td>L10-LVDT</td>
<td>Linear variable displacement transducer</td>
<td>30 mm</td>
<td>LVDT stack row 5 left</td>
</tr>
<tr>
<td>L11-SPOT</td>
<td>String potentiometer</td>
<td>600 mm</td>
<td>Horizontal displacement 2</td>
</tr>
<tr>
<td>L12-SPOT</td>
<td>String potentiometer</td>
<td>600 mm</td>
<td>Horizontal displacement 1</td>
</tr>
<tr>
<td>L13-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Vertical displacement 1</td>
</tr>
<tr>
<td>L14-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Vertical displacement 2</td>
</tr>
<tr>
<td>L15-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Vertical displacement 3</td>
</tr>
<tr>
<td>L16-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Vertical displacement 4</td>
</tr>
<tr>
<td>L17-LPOT</td>
<td>Linear potentiometer</td>
<td>50 mm</td>
<td>Foundation block rotation</td>
</tr>
<tr>
<td>L18-LPOT</td>
<td>Linear potentiometer</td>
<td>50 mm</td>
<td>Foundation block sliding</td>
</tr>
<tr>
<td>L19-LPOT</td>
<td>Linear potentiometer</td>
<td>50 mm</td>
<td>Foundation block rotation</td>
</tr>
<tr>
<td>L20-LPOT</td>
<td>Linear potentiometer</td>
<td>50 mm</td>
<td>Foundation block sliding</td>
</tr>
<tr>
<td>L21-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Foundation block sliding</td>
</tr>
<tr>
<td>L22-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Flange panel (left) sliding</td>
</tr>
<tr>
<td>L23-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Web panel sliding</td>
</tr>
<tr>
<td>L24-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Flange panel (right) sliding</td>
</tr>
</tbody>
</table>
D.5 Precast Connection Testing: Damage Progression

Damage progression photos of test specimens J01, J02 and J03 for the positive and negative loading directions is presented in Figures D.23 and D.24, Figures D.25 and D.26 and Figures D.27 and D.28 respectively. The crack progression was marked on each test specimen on the first positive and negative loading cycle of each displacement increment. Cracking that occurred on the first positive direction loading cycles were marked in red pen, while cracking that occurred on the first negative direction loading cycles were marked in green pen. Force-displacement photo reference diagrams for test specimens J01, J02 and J03 are presented in Figures D.20, D.21 and D.22 respectively.

Figure D.20: Force-displacement photo reference diagram for test specimen J01.

Figure D.21: Force-displacement photo reference diagram for test specimen J02.

Figure D.22: Force-displacement photo reference diagram for test specimen J03.
Figure D.23: Damage progression for test specimen J01 – positive loading.

Note: refer to Figure D.20 for force-displacement diagram showing the location from where the photo series was taken.
Note: refer to Figure D.20 for force-displacement diagram showing the location from where the photo series was taken.

**Figure D.24:** Damage progression for test specimen J01 – negative loading.
Note: refer to Figure D.21 for force-displacement diagram showing the location from where the photo series was taken.

**Figure D.25:** Damage progression for test specimen J02 – positive loading.
(a) cycle 12  (b) cycle 22  (c) cycle 32  
(d) cycle 42  (e) cycle 52  (f) cycle 62

Note: refer to Figure D.21 for force-displacement diagram showing the location from where the photo series was taken.

**Figure D.26:** Damage progression for test specimen J02 – negative loading.
Note: refer to Figure D.22 for force-displacement diagram showing the location from where the photo series was taken.

Figure D.27: Damage progression for test specimen J03 – positive loading.
Note: refer to Figure D.22 for force-displacement diagram showing the location from where the photo series was taken.

Figure D.28: Damage progression for test specimen J03 – negative loading.
D.6 Precast Connection Testing: Welded Stitch Plate Design

This section will provide some additional design guidance for the welded stitch plate design. The section is split into three sub-sections as follows: Section D.6.1, which will outline how to calculate the load distribution between shear studs; Section D.6.2, which will outline how to calculate the load distribution on the welds; and Section D.6.3, which will outline how to calculate the load capacity of the steel stitch plate.

D.6.1 Load Distribution

The horizontal and vertical component of force acting on a group of shear studs can be calculated using the same procedure that would be used when assessing a bolt group in a structural steel connection. Where the vertical load is distributed evenly between the bolts and the moment acting on the bolt group is distributed based on the distance of each respective bolt to the centroid of the bolt group. These equations can be found in most elementary texts on structural steel design. The horizontal (i.e. x-axis) and vertical (i.e. y-axis) force can be calculated for the i-th stud using Equations D.1 and D.2 respectively.

\[
F_{x,i} = \frac{F \cdot y_i}{\sum_{i=1}^{n} \left(x_i^2 + y_i^2\right)} \quad \ldots \text{D.1}
\]

\[
F_{y,i} = \frac{F \cdot x_i}{n} + \frac{F \cdot e}{\sum_{i=1}^{n} \left(x_i^2 + y_i^2\right)} \quad \ldots \text{D.2}
\]

Where:
- \(x_i\) = x-axis distance to the i-th shear stud
- \(y_i\) = y-axis distance to the i-th shear stud
- \(F\) = force
- \(e\) = load eccentricity (i.e. half the distance between the centroid of each stud group)
- \(n\) = number of shear studs

![Figure D.29: Load distribution in shear stud group.](image-url)
D.6.2 Weld Assessment

The weld on each side of the stitch plate connection must resist the vertical shear force transferred across the connection, in addition to the bending moment that is induced. The load in the weld at the different points shown in Figure D.30 can be calculated using Equations D.3 to D.12 [H12]. The design load (i.e. $F$ in Figure D.30) acts in the middle of the stitch plate connection, which means the eccentricity (i.e. $e$ in Figure D.30) would typically be half the thickness of the panel gap. The minimum design weld strength required (i.e. Equation D.10) is in kN/mm when the units of kN are used for force in Equation D.3 and the units of mm are used generally elsewhere.

![Figure D.30: Welded stitch plate weld assessment.](image)

General case:

\[
M = F \times (e + b - x) \quad \ldots \text{D.3}
\]

\[
I_w = 2b + d \quad \ldots \text{D.4}
\]

\[
I_{wp} = I_{wx} + I_{wy} \quad \ldots \text{D.5}
\]

\[
I_{wx} = \frac{d^2(6b + d)}{12} \quad \ldots \text{D.6}
\]

\[
I_{wy} = \frac{b^3(b + 2d)}{3(2b + d)} \quad \ldots \text{D.7}
\]

\[
V_{x,i}^* = \frac{M_{y_i}}{I_{wp}} \quad \ldots \text{D.8}
\]

\[
V_{y,i}^* = \frac{F}{L_w} + \frac{M_{x_i}}{I_{wp}} \quad \ldots \text{D.9}
\]

\[
V_{r,i}^* = \sqrt{V_{x,i}^*^2 + V_{y,i}^*^2} \quad \ldots \text{D.10}
\]

Where:  
- $F$ = vertical shear force (kN)  
- $M$ = in-plane moment (kNm)  
- $V_{r,i}^*$ = design load acting on weld at location $i$ (kN/mm)
For points 1 and 4:

\[ V_{r,i}^* = \sqrt{\left( \frac{Md}{2l_{wp}} \right)^2 + \left( \frac{F}{l_w} + \frac{M(b-x)}{l_{wp}} \right)^2} \]  \hspace{1cm} \ldots \text{D.11}

For points 2 and 3:

\[ V_{r,i}^* = \sqrt{\left( \frac{Md}{2l_{wp}} \right)^2 + \left( \frac{F}{l_w} + \frac{Mx}{l_{wp}} \right)^2} \]  \hspace{1cm} \ldots \text{D.12}

### D.6.3 Steel Stitch Plate Design

The free length of the steel stitch plate is essentially equal to the thickness of the panel gap, which is typically 20 mm. This means the depth of the steel plate is usually many times its 'free length' that bending stresses can be developed across and the critical design check for the steel plate is its shear capacity. The shear capacity and moment capacity of a rectangular steel plate can be calculated in accordance with AS 4100 [X7] using Equations D.13 and D.14 [H12].

\[ \phi V_u = \phi 0.5 f_{sy} dt \]  \hspace{1cm} \ldots \text{D.13}

\[ \phi M_s = \phi f_{sy} \left( \frac{td^2}{4} \right) \]  \hspace{1cm} \ldots \text{D.14}

Where:
- \( \phi = 0.9 \)
- \( f_{sy} = \) yield stress of steel plate
- \( d = \) depth of steel plate
- \( t = \) thickness of steel plate

**Figure D.31:** Design capacity of a rectangular steel plate.
D.7 Precast Connection Testing: Instrumentation Summary

The physical instrumentation used is summarised in Table D.10 for test specimen J01 and J02 and in Table D.11 for test specimen J03.

**Table D.10: Summary of physical instrumentation – test specimen J01 and J02.**

<table>
<thead>
<tr>
<th>Instrumentation</th>
<th>Description</th>
<th>Stroke</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>L01-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Overall displacement</td>
</tr>
<tr>
<td>L02-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Top connection</td>
</tr>
<tr>
<td>L03-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Top connection</td>
</tr>
<tr>
<td>L04-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Bottom connection</td>
</tr>
<tr>
<td>L05-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Bottom connection</td>
</tr>
</tbody>
</table>

**Table D.11: Summary of physical instrumentation – test specimen J03.**

<table>
<thead>
<tr>
<th>Instrumentation</th>
<th>Description</th>
<th>Stroke</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>L01-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Overall displacement</td>
</tr>
<tr>
<td>L02-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Top corbel displacement</td>
</tr>
<tr>
<td>L03-LPOT</td>
<td>Linear potentiometer</td>
<td>50 mm</td>
<td>Corbel gap displacement</td>
</tr>
<tr>
<td>L04-SPOT</td>
<td>String potentiometer</td>
<td>300 mm</td>
<td>Corbel gap crack size</td>
</tr>
<tr>
<td>L05-LPOT</td>
<td>Linear potentiometer</td>
<td>50 mm</td>
<td>Corbel gap crack size</td>
</tr>
</tbody>
</table>