Document Status and Amendments

<table>
<thead>
<tr>
<th>Version</th>
<th>Date</th>
<th>Purpose/ Amendment Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2016_SB</td>
<td>30 June 2016</td>
<td>Draft for Sector Briefings</td>
</tr>
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<td>2016_C</td>
<td>10 Oct 2016</td>
<td>Revised draft to accompany consultation proposals for the regulations and EPB methodology under the Building (Earthquake-prone Buildings) Amendment Act 2016</td>
</tr>
</tbody>
</table>

This document is intended to be referenced by the Earthquake Prone Buildings (EPB) Methodology being developed under the provisions of the Building (Earthquake-prone Buildings) Amendment Act 2016. It is also intended to be endorsed by MBIE for use as guidance under section 175 of the Building Act 2004 to the extent that it assists practitioners and territorial authorities in complying with that Act.

Document Access

This draft document may be downloaded from www.EQ-Assess.org.nz in parts:

1. Part A – Assessment Objectives and Principles
2. Part B – Initial Seismic Assessment
3. Part C – Detailed Seismic Assessment

Updates will be notified on the above website.

The document is expected to be published before the Act comes into force, when the regulations and EPB Methodology associated with the Building (Earthquake-prone Buildings) Amendment Act 2016 come into force.

Document Management and Key Contact

This document is managed jointly by the Ministry of Business, Innovation and Employment, the Earthquake Commission, the New Zealand Society for Earthquake Engineering, the Structural Engineering Society and the New Zealand Geotechnical Society.

Please use the feedback forms on www.EQ-Assess.org.nz to provide feedback or to request further information about these draft Guidelines.
Acknowledgements

These Guidelines were prepared during the period 2014 to 2016 with extensive technical input from the following members of the Project Technical Team:

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</tbody>
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Funding for the development of these Guidelines was provided by the Ministry of Business, Innovation and Employment and the Earthquake Commission.
Contents

C5. Concrete Buildings .............................................. C5-1
C5.1 General .................................................................. C5-1
  C5.1.1 Scope and outline of this section ....................... C5-1
  C5.1.2 Useful publications ........................................ C5-3
  C5.1.3 Notation .......................................................... C5-4
C5.2 Typical Concrete Building Practices in New Zealand .......... C5-8
  C5.2.1 General .......................................................... C5-8
  C5.2.2 1920s to 1950s: early years of seismic design .......... C5-8
  C5.2.3 1960s to mid-1970s: advent of structural ductility ... C5-9
  C5.2.4 Mid-1970s onwards: modern seismic design .......... C5-10
C5.3 Observed Behaviour of Reinforced Concrete Buildings in
  Earthquakes ............................................................. C5-11
  C5.3.1 General .......................................................... C5-11
  C5.3.2 Non-ductile columns ......................................... C5-12
  C5.3.3 Damage observations following the Canterbury
            earthquakes .................................................. C5-14
C5.4 Material Properties and Testing ................................ C5-39
  C5.4.1 General .......................................................... C5-39
  C5.4.2 Concrete ........................................................ C5-40
  C5.4.3 Reinforcing steel rebars .................................... C5-42
  C5.4.4 Cold wire mesh ............................................... C5-45
C5.5 Element Probable Capacities .................................... C5-46
  C5.5.1 General approach ............................................ C5-46
  C5.5.2 Beam capacity ............................................... C5-49
  C5.5.3 Columns ........................................................ C5-62
  C5.5.4 Beam-column joints ......................................... C5-73
  C5.5.5 Structural walls .............................................. C5-84
  C5.5.6 Concrete floor diaphragms ................................ C5-95
C5.6 Global Capacity of Moment Resisting Concrete Frame Buildings...... C5-102
  C5.6.1 Evaluation of the hierarchy of strength and sequence of events
            for a beam-column joint subassembly ................... C5-102
  C5.6.2 Effect of varying axial load on joint capacity ........ C5-104
  C5.6.3 Upper and lower bounds of base shear capacity and force-
            displacement curves ........................................ C5-106
C5.7 Global Capacity of Wall Buildings ................................ C5-108
  C5.7.1 General ........................................................ C5-108
  C5.7.2 Evaluation approach ......................................... C5-109
C5.8 Global Capacity of Dual Frame-Wall Concrete Buildings .......... C5-110
  C5.8.1 General ........................................................ C5-110
  C5.8.2 Derivation of global force-displacement capacity curve ... C5-112
C5.9 Improving the Seismic Performance of Concrete Buildings ........ C5-116
References .............................................................. C5-117

Appendix C5A : Key Milestones in the Evolution of New Zealand Concrete
  Design Standards, and Historical Concrete Property Requirements
  and Design Specifications in New Zealand ........................ C5-1

Appendix C5B : Historical Requirements for Concrete Strength Testing in
  New Zealand ............................................................ C5-11
Appendix C5C : Material Test Methods................................................................. C5-12
Appendix C5D : History of New Zealand Reinforcement Requirements ........ C5-16
Appendix C5E : Diaphragm Grillage Modelling Methodology ......................... C5-39
Appendix C5F : Deformation Capacity of Precast Concrete Floor Systems ...... C5-46
Appendix C5G : Buckling of Vertical Reinforcement and Out-of-Plane
Instability in Shear Walls...................................................................................... C5-56

The current version of this section has not yet been fully edited and
co-ordinated with other sections of Part C.
C5. Concrete Buildings

C5.1 General

C5.1.1 Scope and outline of this section

This section provides guidelines for performing a DSA for existing reinforced concrete (RC) buildings from the material properties to section, component, subassembly, and ultimately the system level. Unreinforced concrete structures are not addressed.

The overall aim is to provide assessors with:

- an understanding of the underlining issues associated with the seismic response of RC buildings (including the presence of inherent vulnerabilities or weaknesses), and
- a set of assessment tools based on different levels of complexity (not necessarily corresponding to different levels of reliability) for the detailed seismic assessment (DSA) of the behaviour of RC buildings, with particular reference to evaluation of %NBS.

Note:

This section is based on the latest information and knowledge relating to the seismic behaviour of existing RC buildings which has been developed and gained over the last 15 years at both the national and international level. It also draws on international standards and guidelines on seismic assessment and strengthening/retrofitting, with the aim of adapting and integrating best practice to best suit New Zealand conditions.

Increased knowledge in relation to RC buildings has been obtained through extensive experimental and analytical/numerical investigations, and also through damage observations and lessons learned following major earthquakes. In particular, there have been two significant projects relating to New Zealand construction practice:

- the Foundation of Research Science and Technology (FRST) research project ‘Retrofit Solutions for New Zealand Multi-storey Buildings’, which was carried out jointly by the University of Canterbury and University of Auckland from 2004 to 2010, and
- the ‘SAFER Concrete Technology’ Project (2011-2015), funded by the Natural Hazard Research Platform (NHRP).

These projects have provided very valuable evidence-based information on the expected seismic performance of concrete buildings designed and constructed according to New Zealand practice and Building Code provisions. (Refer, for an overview of these findings, Pampanin 2009 and, for more details, Marriott, 2009; Kam, 2011; Akguzel, 2011; Genesio, 2011; and Quintana-Gallo, 2014.)

More recently, the Canterbury earthquake sequence of 2010-2011 has represented a unique “open-air laboratory” and an important source of information for assessing and evaluating the actual seismic performance of New Zealand RC buildings of different structural type, age, construction practice and design details.

Recent experience has highlighted a number of key structural weaknesses and failure mechanisms, either at an element level or at a global system level. It has not only confirmed
that pre-1970s RC buildings – as expected – have a potentially high inherent seismic vulnerability, but also that some modern (e.g. post-1980s) RC buildings can be expected to perform poorly. In some cases, this has led to catastrophic collapses or “near misses”. This has been a wake-up call as it has identified a “new generation” of potentially vulnerable buildings that need to be scrutinised with care.

This section attempts to capture these new learnings and provide up to date procedures for evaluating the vulnerability of existing RC buildings and for determining their seismic rating. It dedicates specific effort to describing, both qualitatively and quantitatively, key aspects of the local and global mechanisms and their impact on the building response. This is to provide practising engineers with a more holistic understanding of the overall building capacity and expected performance, which is essential when determining the seismic rating for a building.

Note:

Most RC buildings designed post-1976 can be expected to have a relatively low probability of collapse under ULS level earthquake shaking.

However, some of these buildings can still have structural weaknesses – even severe structural weaknesses, such as non-ductile gravity columns with low drift capacity – which could lead to a progressive and catastrophic collapse in severe earthquakes.

This section covers in turn:

- typical building practices, structural deficiencies and observed behaviour of RC buildings in earthquakes (refer to Sections C5.2 to C5.3)
- material properties and testing, component capacities and global system capacities (refer to Sections C5.4 to C5.8), and
- brief comments on improving RC buildings (refer to Section C5.9).

Sections C5.2 to C5.3, referred to above, provide important context for any assessment of RC buildings and include findings from the Canterbury earthquake sequence of 2010-11. An appreciation of the observed behaviour of a building in the context of its age and the detailing present is considered an essential part of assessing its seismic rating. It is recommended that assessors become familiar with the material in these sections before conducting an assessment.

Given their importance in the overall behaviour of a building system, as emphasised by the lessons learnt in recent earthquakes, RC floor/diaphragms and their interactions with the main vertical lateral load-resisting systems are covered in some detail in Section C5.5.6.

This material should be read in conjunction with the more general guidance outlined in Section C2.

Appendices include summaries of:

- evolution of New Zealand design standards, (refer to Appendix C5A)
- historical concrete property requirements, design specifications and strength testing in New Zealand (refer to Appendix C5B)
- material test methods for concrete and reinforcing steel (refer to Appendix C5C)
• the evolution of steel reinforcing standards in New Zealand, including reference values for the mechanical properties of the reinforcing steel depending on the age of construction (refer to Appendix C5D).

Further appendices include guidance for:
• diaphragm grillage modelling (refer to Appendix C5E)
• assessing the deformation capacity of precast concrete floor systems (refer to Appendix C5F)
• assessing the buckling of vertical reinforcement in shear walls (refer to Appendix C5G).

Note:
The impact of masonry infills on the performance of the primary structural systems is covered in Section C7. The effects of Soil-Structure Interaction (SSI) in terms of seismic performance, modifications of demand and development of mixed mechanisms are discussed in Section C4.

C5.1.2 Useful publications

A short list of key publications follows. A more comprehensive list is provided at the end of this section and referenced throughout.

ASCE 41-13 (2014). *Seismic Evaluation and Retrofit of Existing Buildings*, American Society of Civil Engineers and Structural Engineering Institute, Reston, Virginia, USA

ATC 78-3 (2015). *Seismic Evaluation of Older Concrete Frame Buildings for Collapse Potential*, Applied Technology Council (ATC), Redwood City, California, USA

FEMA P-58 (2012). *Seismic Performance Assessment of Buildings*, Applied Technology Council (ATC), Redwood City, California, USA


NTC (2008), *Norme tecniche per le costruzioni*, (Code Standard for Constructions), (In Italian), Ministry of Infrastructure and Transport, MIT, Rome, Italy


## C5.1.3 Notation

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Meaning</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$</td>
<td>Depth of the compression stress block ($=\beta_c$)</td>
</tr>
<tr>
<td>$A_{bb}$</td>
<td>Displacement at the onset of bar buckling</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross section area of the column</td>
</tr>
<tr>
<td>$A_r$</td>
<td>Wall aspect ratio</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Area of reinforcement in tension</td>
</tr>
<tr>
<td>$A_s'$</td>
<td>Area of reinforcement in compression</td>
</tr>
<tr>
<td>$A_{sp}$</td>
<td>Area of spiral or circular hoop bar</td>
</tr>
<tr>
<td>$A_{st}$</td>
<td>Area of transverse reinforcement parallel to the applied shear</td>
</tr>
<tr>
<td>$A_{st}$</td>
<td>Area of transverse reinforcement parallel to the applied shear</td>
</tr>
<tr>
<td>$A_t$</td>
<td>Area of the transverse stirrups</td>
</tr>
<tr>
<td>$A_v$</td>
<td>Area of transverse shear reinforcement at spacing $s$</td>
</tr>
<tr>
<td>$A_{s,\text{eff}}$</td>
<td>Area of the effective steel of the slab</td>
</tr>
<tr>
<td>$b_o$</td>
<td>Effective width of the spandrel for torsion</td>
</tr>
<tr>
<td>$b_b$</td>
<td>Beam width</td>
</tr>
<tr>
<td>$b_c$</td>
<td>Column width</td>
</tr>
<tr>
<td>$b_{\text{core}}$</td>
<td>Width of column core, measured from centre to centre of the peripheral transverse reinforcement in the web</td>
</tr>
<tr>
<td>$b_{\text{eff}}$</td>
<td>Effective width of the slab</td>
</tr>
<tr>
<td>$b_j$</td>
<td>Effective width of the joint</td>
</tr>
<tr>
<td>$b_w$</td>
<td>Web width</td>
</tr>
<tr>
<td>$b_{w}$</td>
<td>Width of beam web</td>
</tr>
<tr>
<td>$C$</td>
<td>Neutral axis depth</td>
</tr>
<tr>
<td>$C'$</td>
<td>Resultant of compression stresses in compression reinforcement</td>
</tr>
<tr>
<td>$D$</td>
<td>Section effective depth</td>
</tr>
<tr>
<td>$d''$</td>
<td>Depth of the concrete core of the column measured in the direction of the shear force for rectangular hoops, and the diameter of the concrete core for spirals or circular hoops</td>
</tr>
<tr>
<td>$s$</td>
<td>Spacing of transverse shear reinforcement</td>
</tr>
<tr>
<td>$c_0$</td>
<td>Cover to longitudinal bars</td>
</tr>
<tr>
<td>$c_u$</td>
<td>Neutral axis depth at ultimate curvature</td>
</tr>
<tr>
<td>$d_b$</td>
<td>Average diameter of longitudinal reinforcement</td>
</tr>
<tr>
<td>Symbol</td>
<td>Meaning</td>
</tr>
<tr>
<td>--------</td>
<td>---------</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Steel elastic modulus</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>Probable concrete compressive strength</td>
</tr>
<tr>
<td>$f'_{cc}$</td>
<td>Probable confined concrete compressive strength</td>
</tr>
<tr>
<td>$f_{st}$</td>
<td>Stress in the steel related to the maximum tensile strain in the first part of the cycle</td>
</tr>
<tr>
<td>$f_u$</td>
<td>Probable ultimate strength of the longitudinal reinforcement</td>
</tr>
<tr>
<td>$f_v$</td>
<td>Normal stress in the vertical direction</td>
</tr>
<tr>
<td>$f_y$</td>
<td>Probable yielding strength of the longitudinal reinforcement</td>
</tr>
<tr>
<td>$f_{y/slab}$</td>
<td>Yielding stress of the slab steel in tension</td>
</tr>
<tr>
<td>$f_{yt}$</td>
<td>Yielding stress of the transverse steel</td>
</tr>
<tr>
<td>$f_{yt}$</td>
<td>Probable yield strength of the transverse reinforcement</td>
</tr>
<tr>
<td>$H$</td>
<td>Height of the member</td>
</tr>
<tr>
<td>$h_b$</td>
<td>Beam height</td>
</tr>
<tr>
<td>$h_c$</td>
<td>Column height</td>
</tr>
<tr>
<td>$h_{cr}$</td>
<td>Vertical height of inclined crack</td>
</tr>
<tr>
<td>$h_t$</td>
<td>Height of the transverse beam or spandrel</td>
</tr>
<tr>
<td>$h_w$</td>
<td>Wall height</td>
</tr>
<tr>
<td>$J_d$</td>
<td>Internal couple lever arm</td>
</tr>
<tr>
<td>$K$</td>
<td>Shear stress degradation factor</td>
</tr>
<tr>
<td>$K_d$</td>
<td>Neutral axis depth when tension steel reaches the strain at first yield, $\varepsilon_y$</td>
</tr>
<tr>
<td>$k_j$</td>
<td>Coefficient for calculating the shear capacity of a joint</td>
</tr>
<tr>
<td>$k_{lp}$</td>
<td>Coefficient related to the plastic hinge calculation</td>
</tr>
<tr>
<td>$k_{wall}$</td>
<td>Shear coefficient related to concrete mechanism</td>
</tr>
<tr>
<td>$l_b$</td>
<td>Half of the length of the beam</td>
</tr>
<tr>
<td>$L_c$</td>
<td>Shear span, distance of the critical section from the point of contra flexure</td>
</tr>
<tr>
<td>$l_c$</td>
<td>Total length of the column</td>
</tr>
<tr>
<td>$l_{cr}$</td>
<td>Horizontal length of inclined crack</td>
</tr>
<tr>
<td>$l_d$</td>
<td>Theoretical development length</td>
</tr>
<tr>
<td>$l_{d,prov}$</td>
<td>Provided lap length</td>
</tr>
<tr>
<td>$l_{d,req}$</td>
<td>Required lap length</td>
</tr>
<tr>
<td>$L_p$</td>
<td>Plastic hinge length</td>
</tr>
<tr>
<td>$L_{sp}$</td>
<td>Strain penetration length</td>
</tr>
<tr>
<td>$l_w$</td>
<td>Wall length</td>
</tr>
<tr>
<td>Symbol</td>
<td>Meaning</td>
</tr>
<tr>
<td>------------</td>
<td>----------------------------------------------------------------</td>
</tr>
<tr>
<td>$M$</td>
<td>Bending moment</td>
</tr>
<tr>
<td>$M_b$</td>
<td>Moment in the beam (at the interface with the column)</td>
</tr>
<tr>
<td>$M_{col}$</td>
<td>Equivalent moment in the column (at the level of the top face of the beam)</td>
</tr>
<tr>
<td>$M_f$</td>
<td>Residual moment capacity of an element</td>
</tr>
<tr>
<td>$M_{lap}$</td>
<td>Moment capacity of a lap splice</td>
</tr>
<tr>
<td>$M_n$</td>
<td>Probable flexural moment capacity of an element</td>
</tr>
<tr>
<td>$M_{p,wall}$</td>
<td>Wall probable flexural strength</td>
</tr>
<tr>
<td>$N$</td>
<td>Axial load</td>
</tr>
<tr>
<td>$N^*$</td>
<td>Total axial load: gravity plus seismic.</td>
</tr>
<tr>
<td>$p_t, p_c$</td>
<td>Tensile and compressive average principal stresses in the joint panel</td>
</tr>
<tr>
<td>$S_n$</td>
<td>Nominal strength capacity</td>
</tr>
<tr>
<td>$S_o$</td>
<td>Overstrength capacity</td>
</tr>
<tr>
<td>$S_{prob}$</td>
<td>Probable strength capacity</td>
</tr>
<tr>
<td>$s_t$</td>
<td>Spacing in between stirrups in the spandrel</td>
</tr>
<tr>
<td>$T$</td>
<td>Resultant of tension stresses in tension reinforcement</td>
</tr>
<tr>
<td>$V$</td>
<td>Shear</td>
</tr>
<tr>
<td>$V$</td>
<td>Maximum nominal shear stress</td>
</tr>
<tr>
<td>$V_b$</td>
<td>Shear force in the beam</td>
</tr>
<tr>
<td>$V_c$</td>
<td>Shear resisted by the concrete mechanisms</td>
</tr>
<tr>
<td>$V_c$</td>
<td>Shear force in the column</td>
</tr>
<tr>
<td>$v_c$</td>
<td>Nominal shear stress carried by concrete mechanism</td>
</tr>
<tr>
<td>$V_{c,wall}$</td>
<td>Shear resisted by the concrete mechanisms</td>
</tr>
<tr>
<td>$v_{ch}$</td>
<td>Nominal horizontal joint shear stress carried by a diagonal compressive strut mechanism crossing joint</td>
</tr>
<tr>
<td>$V_{jh}$</td>
<td>Average shear stress in the joint panel</td>
</tr>
<tr>
<td>$V_{jh}$</td>
<td>Horizontal joint shear force</td>
</tr>
<tr>
<td>$V_n$</td>
<td>Shear resisted as a result of the axial compressive load</td>
</tr>
<tr>
<td>$V_{n,wall}$</td>
<td>Shear resisted as a result of the axial compressive load</td>
</tr>
<tr>
<td>$V_p$</td>
<td>Probable shear strength capacity of an element</td>
</tr>
<tr>
<td>$V_{p,wall}$</td>
<td>Wall probable shear strength</td>
</tr>
<tr>
<td>$V_{pjh}$</td>
<td>Probable horizontal joint shear force</td>
</tr>
<tr>
<td>$v_s$</td>
<td>Shear resisted by the transverse shear reinforcement</td>
</tr>
<tr>
<td>$V_{s,wall}$</td>
<td>Shear resisted by the horizontal transverse shear reinforcement</td>
</tr>
<tr>
<td>$\alpha'$</td>
<td>Shear coefficient related to section aspect ratio</td>
</tr>
<tr>
<td>Symbol</td>
<td>Meaning</td>
</tr>
<tr>
<td>----------</td>
<td>-------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$\alpha$, $\beta$</td>
<td>Stress block parameters</td>
</tr>
<tr>
<td>$\alpha'_{\text{wall}}$</td>
<td>Shear coefficient related to section aspect ratio</td>
</tr>
<tr>
<td>$\beta'$</td>
<td>Shear coefficient related to longitudinal reinforcement ratio</td>
</tr>
<tr>
<td>$\beta'_{\text{wall}}$</td>
<td>Shear coefficient related to longitudinal reinforcement ratio</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Inclination angle of axial load compressive truss</td>
</tr>
<tr>
<td>$\gamma_{bb,l,w}$</td>
<td>Wall core length</td>
</tr>
<tr>
<td>$\Delta_p$</td>
<td>Plastic displacement</td>
</tr>
<tr>
<td>$\Delta_u$</td>
<td>Ultimate displacement</td>
</tr>
<tr>
<td>$\Delta_y$</td>
<td>Yielding displacement</td>
</tr>
<tr>
<td>$\delta^*_p$</td>
<td>Plastic displacement at the onset of bar buckling</td>
</tr>
<tr>
<td>$\varepsilon^*_b$</td>
<td>Tensile strain in the steel at zero stress</td>
</tr>
<tr>
<td>$\varepsilon^*_{\text{cm}}$</td>
<td>Concrete strain at the onset of bar buckling (reversed actions)</td>
</tr>
<tr>
<td>$\varepsilon^*_{\text{tp}}$</td>
<td>Steel plastic strain at the onset of bar buckling</td>
</tr>
<tr>
<td>$\varepsilon_{\text{cu}}$</td>
<td>Concrete ultimate compressive strain</td>
</tr>
<tr>
<td>$\varepsilon_s$</td>
<td>Tension steel strain</td>
</tr>
<tr>
<td>$\varepsilon_{\text{s,cr}}$</td>
<td>Steel tensile strain at the onset of bar buckling (cyclic actions)</td>
</tr>
<tr>
<td>$\varepsilon_{\text{sh}}$</td>
<td>Strain at the end of the yielding plateau</td>
</tr>
<tr>
<td>$\varepsilon_{\text{st}}$</td>
<td>Maximum tensile strain in the steel in the first part of the cycle</td>
</tr>
<tr>
<td>$\varepsilon_{\text{su,b}}$</td>
<td>Steel tensile strain at the onset of bar buckling (monotonic actions)</td>
</tr>
<tr>
<td>$\varepsilon_{\text{su}}$</td>
<td>Steel ultimate tensile strain</td>
</tr>
<tr>
<td>$\varepsilon_y$</td>
<td>Strain at first yield of the longitudinal tension reinforcement</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Rotation (or drift ratio)</td>
</tr>
<tr>
<td>$\theta_{\text{cr}}$</td>
<td>Average cracking angle</td>
</tr>
<tr>
<td>$\theta_p$</td>
<td>Plastic rotation (or drift ratio)</td>
</tr>
<tr>
<td>$\theta_u$</td>
<td>Ultimate rotation (or drift ratio)</td>
</tr>
<tr>
<td>$\theta_y$</td>
<td>Yielding rotation (or drift ratio)</td>
</tr>
<tr>
<td>$\mu_{\Delta}$</td>
<td>Displacement ductility</td>
</tr>
<tr>
<td>$\mu_{\Delta c}$</td>
<td>Displacement ductility capacity</td>
</tr>
<tr>
<td>$\mu_{\Delta d}$</td>
<td>Displacement ductility demand</td>
</tr>
<tr>
<td>$\mu_{\phi}$</td>
<td>Curvature ductility</td>
</tr>
<tr>
<td>$\rho_{\text{eff}}$</td>
<td>Effective confinement ratio</td>
</tr>
<tr>
<td>$\rho_{\ell}$</td>
<td>Longitudinal reinforcement ratio</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>Volume of transverse reinforcement to volume of concrete core ratio</td>
</tr>
</tbody>
</table>
### C5.2 Typical Concrete Building Practices in New Zealand

#### C5.2.1 General

Construction methods for RC buildings in New Zealand have changed significantly over the years since their first appearance in the 1920s. The evolution of construction methods matches the evolution of the relevant codes and standards in line with increasing understanding of the behaviour of these buildings in earthquakes.

An understanding of the development of seismic design provisions for RC buildings is relevant for the assessor as it often provides valuable insight into why certain detailing decisions were made and the possible presence of SSWs (refer to Section C1), particularly where deformation capacity might be limited.

Developments in the design requirements for RC buildings and the corresponding evolution of loading standards are summarised in Appendix C5A, along with some pointers on what to look for in RC buildings of the corresponding eras. An overview of the key historical code developments is given in this section.

**Note:**
For a more detailed comparison of New Zealand standards used for seismic design of RC buildings refer to Fenwick and MacRae, 2009. A summary of the evolution of earthquake engineering codified requirements in New Zealand has also been provided by Kam and Pampanin (2002).

#### C5.2.2 1920s to 1950s: early years of seismic design

The first known New Zealand publication on earthquake design was written by C. Reginald Ford (Ford, 1926) in 1926, several years before the 7.8 magnitude Napier earthquake of 1931 that changed New Zealand construction practice dramatically. Ford’s description drew heavily from the state of knowledge and lessons following the San Francisco (1906) and Kanto, Japan (1923) earthquakes. However, the significant loss of lives and devastation following the 1931 Napier earthquake (Dewell, 1931) provided the government with the
impetus to legislate building construction in relation to earthquake resistance. A Building Regulations Committee was set up and reported on a draft earthquake building by-law, which was presented to the New Zealand Parliament in June 1931 (Cull, 1931). This draft building by-law was subsequently published by New Zealand standards as the 1935 New Zealand Standard (NZS) Model Building By-Law (NZSS 95:1935, 1935) and the 1939 NZS Code of Building By-Laws (NZSS 95:1939, 1939).

The 1935 by-law (NZSS 95:1935, 1935) was not compulsory and depended on adoption by local territorial authorities. There were no specific recommendations for the design of concrete buildings. However, it is interesting to note that 135 degree hooks were already shown for stirrups in reinforced construction (clause 409 of NZSS 95).

The 1955 revision of the NZS Standard Model Building By-Law (NZSS 95:1955) introduced changes but lacked significant improvement in terms of seismic structural detailing. For example, while it gave explicit definitions for deformed bars (which were only introduced in New Zealand in the mid-1960s) and plain round bars, it only specified 10% higher allowable bond stresses for deformed bars. The provisions for shear resistance of concrete elements were tightened and the requirement of 135° anchorage for stirrups was included. However, no other specific seismic details for reinforced concrete structures were specified.

C5.2.3 1960s to mid-1970s: advent of structural ductility

The NZS 1900:1964 code (NZS 1900.8-64, 1964, NZS 1900.9-64, 1964) was a significant evolution from its predecessors. It showed increased understanding of RC seismic design, and was also based on best international practice and knowledge (ACI318-63, 1963, CEB-1964, 1964).

This code introduced the concept of structural ductility with the stated assumption of 5-10% damping for structural ductility $\mu = 4$ for RC structures. However, no provision for ductile RC detailing or modern capacity design considerations (yet to be developed) was included.

Notably, NZS 1900:1964 was still based on the working (allowable) stress concept for member design while the international trend, in particular for RC design provisions or Model Codes (fib), was starting to move towards the introduction of limit state design concepts (ACI318-63, 1963; CEB-1964, 1964).

In 1961, work by Blume, Newmark and Corning (Blume, et al., 1961) had pioneered the concept of ductile RC buildings and introduced detailing for ductile RC elements. As the 1960s and 1970s progressed, there were significant developments in earthquake engineering internationally, as summarised in the 1966-1973 Structural Engineers Association of California (SEAOC) recommendations (SEAOC, 1966; SEAOC, 1973) and the 1971 ACI-318 concrete code (ACI 318-71, 1971). The need for beam-column joint seismic design, different ductility coefficient for different lateral-resistant systems and ductile RC detailing were identified in these documents.

However, the 1971 ACI-318 code (ACI 318-71, 1971) did not contain any of the capacity design provisions which were developed in New Zealand in the late 1960s-1970s (Park and Paulay, 1975). As a result, without explicit design for lateral-force resistance, for example, buildings constructed before the NZSS 95:1955 provisions were introduced – or pre-1970s
RC frames more generally – are unlikely to have sufficient lateral strength capacity or adequate lateral stiffness because of small column dimensions (proportioned primarily for gravity loads).


Park and Paulay’s seminal publication of 1975 (Park and Paulay, 1975) outlined many concepts of modern seismic RC design and detailing, including a rigorous design procedure of RC frames under the capacity design philosophy and quantification of the ductility capacity of RC beam, column, wall and joint elements. These innovations were quickly disseminated in New Zealand engineering practice and building standards (NZS 3101:2006, 2006) from the mid-1970s onwards.

**C5.2.4 Mid-1970s onwards: modern seismic design**

The introduction of the NZS 4203:1976 loading standard represented a quantum change in the approach to seismic design. The limit state approach using defined Ultimate Limit State (ULS) and Serviceability Limit State (SLS) was codified as the preferred design method over the working stress approach. Ductility was required to be explicitly allowed for (as per the 1966 SEAOC recommendations). Structures without any ductile detailing were required to be designed for higher seismic loading.

In the same period, the provisional NZS 3101 concrete standard, published in 1972 (NZS 3101:1970P, 1970) also adopted many parts of the 1971 ACI-318 code (ACI318-71, 1971) and some recommendations from the draft of (Park and Paulay, 1975). It introduced some detailing of plastic hinge regions with a focus on shear reinforcement, lapping of bars and column confinement.

However, it was not until the revamp of the New Zealand loading code NZS 4203 in 1976, the update of the ACI-318 code in 1977 and the various drafts of the 1982 edition of the NZS 3101 concrete design standard (NZS 3101:1982, 1982) that modern seismic design for RC buildings was fully codified in New Zealand.

NZS 3101:1982 provided improved requirements in the detailing of plastic hinge regions, including shear, confinement and anti-buckling reinforcement. Lapped bars were not permitted at any floor levels in columns where there was a possibility of yielding. Column ties were required to be anchored by 135 degrees in cover concrete. Improved methods of determining spacing of transverse reinforcement for seismic columns were provided. A strong-column weak beam mechanism was explicitly specified in the commentary of this standard, with requirements to account for overstrength moments including flange effects from the slab.

As an example of key improvements between 1982 and 1995, both in conceptual design and required details, a potential “loophole” in the 1982 code relating to the design of gravity columns (now typically referred to as pre-1995 “non-ductile” columns) was removed when improved provisions were included in NZS 3101:1995 (refer to Section C5.5.3 for more details).

Note:
The late 1970s through to the 1990s represent a period when the knowledge of seismic performance of buildings improved significantly. As a result, the development of standards over this period often lagged behind the published research. In New Zealand the Bulletin of the New Zealand National Society for Earthquake Engineering, BNZSEE, published a number of papers that were the precursor of provisions which ultimately translated into design requirements. Designers often incorporated these refinements into their designs long before the provisions were cited in the standards.

For this reason any assumption regarding detailing that are based solely on the date of design/construction should be approached with care. Non-invasive and/or intrusive investigations will be required to confirm such assumptions when these are found to be key to the assessed behaviour of the building.

### C5.3 Observed Behaviour of Reinforced Concrete Buildings in Earthquakes

#### C5.3.1 General

Extensive experimental and analytical investigations into the seismic vulnerability and response/performance of RC buildings, together with observations of damage in past earthquakes (including the Canterbury earthquake sequence of 2010/11) have highlighted a series of typical structural deficiencies in RC buildings.

These include:
- inadequate transverse reinforcement for shear and confinement in potential plastic hinge regions
- insufficient transverse reinforcement in beam-column joint core regions
- insufficient and inadequate detailing of column longitudinal and transverse reinforcement
- inadequate anchorage detailing in general, for both longitudinal and transverse reinforcement
- insufficient lap splices of column reinforcement just above the floor or at the foundation level, or of beam reinforcement in regions where the gravity moments are high
- insufficient shear, anti-buckling and confining/restraining reinforcement in wall systems
- insufficient longitudinal reinforcement ratio in walls, combined with higher than expected tensile strength in the concrete, leading to single crack opening when compared to a spread plastic hinge, resulting in failure in tension of the rebars
- inadequate capacity of the foundations to account for overturning moment caused by lateral loading
• lower quality of materials (concrete and steel) when compared to current practice; in particular:
  – use of low grade plain round (smooth) bars for both longitudinal (until the mid-1960s) and transverse reinforcement
  – low-strength concrete (below 20-25 MPa, and, in extreme cases, below 10 MPa)
• potential brittle failure mechanisms at both local and global level due to interaction with spandrel beams, masonry infills, façades causing shear failure in columns (due to short/captive column effects) and/or potential soft-storey mechanisms
• lack of (horizontal and vertical) displacement compatibility considerations between the lateral load resisting systems (either frames, walls or a combination of these), the floor-diaphragms and gravity load bearing systems (e.g. non-ductile columns with limited confinement details and drift capacity)
• inadequate design of diaphragm actions and connection detailing; particularly in the case of precast concrete floor systems which became common from the 1980s onwards
• inadequate protection against punching shear between columns and flat-slab connections
• plan and vertical irregularity, resulting in unexpected amplification and concentration of demands on beams, walls and columns
• limited and inadequate consideration of bidirectional loading effect on critical structural elements (e.g. columns, walls, or beam-column joints), and
• lack of, or inadequate consideration of, capacity design principles. While this is more typical of pre mid-1970s RC buildings (before the introduction of NZS 4203:1976 and the capacity design concept itself), it can also arise in later buildings as this concept was under continuous refinement in further generations of building standards.

It is worth noting that often structural deficiencies are not isolated. Brittle failure mechanisms can be expected either at local level (e.g. shear failure in the joints, columns or beams) or global level (e.g. soft storey mechanisms). The presence of multiple structural deficiencies and lack of an alternative robust load path – i.e. lack of redundancy/robustness – can trigger progressive collapse with catastrophic consequences, as evident in the 22 February 2011 Christchurch (Lyttleton) earthquake.

**Note:**
While the deficiencies listed above have been shown to reduce the performance of RC buildings, noncompliance with current standards is not necessarily an indication of inadequate performance when compared against the minimum requirements of the Building Code. The effect of the deficiencies on the building behaviour and therefore its earthquake rating will depend on their location and criticality and the assessed impact of failure on life safety.

**C5.3.2 Non-ductile columns**

Gravity columns are common in structural systems that contain shear walls, seismic frames, or a combination of both as the lateral load resisting system. These columns are generally required to support often significant areas of floor, while not being relied upon to contribute to the strength of the lateral system. In order to perform this function they must remain capable of carrying axial load while undergoing the required lateral displacements of the structural system.
If these displacements are particularly large and/or the axial loads in the columns are large, there is the potential for the gravity columns to be a severe structural weakness (SSW) with potentially catastrophic consequences.

The poor performance of reinforced concrete columns with inadequate detailing, such as inadequate transverse reinforcement, lap-splices in the plastic hinge region and possibly longitudinal rebars ‘cranked’ at the end of the lap splices, has been observed in various past earthquakes (refer to Figure C5.1) and investigated in recent literature (in particular, Boys et al., 2008; Elwood and Moehle, 2005; and Kam et al., 2011).

(a) Indian Hills Medical Centre (1994 Northridge earthquake)  
(b) Olive View Hospital (1971 San Fernando earthquake)

Figure C5.1: Examples of failure of inadequately reinforced columns in past earthquakes

In addition to older (pre-1970s) details, which were expected to have a number of deficiencies, a potential loophole in the NZS 3101:1982 design standard was identified for the detailing of columns designed according to post-1982 and pre-1995 code specifications.

**Note:**

Experimental tests conducted at the University of Canterbury by Boys et al. in 2008 (and therefore before the Canterbury earthquake sequence of 2010/11), which reflected New Zealand construction and design detailing, highlighted the potentially high vulnerability of gravity columns with inadequate/poor detailing to sustain lateral displacements.

These tests comprised both unidirectional and bidirectional loading testing regimes. They showed the low displacement/drift capacity of such columns, which was exacerbated by a bidirectional loading regime (more realistically representing the actual response of a building under a ground motion).

Figure C5.2 presents examples of axial-shear failure of non-ductile gravity columns simulated in this laboratory testing under unidirectional cyclic loading.

In general, the (limited) experimental tests that were carried out confirmed that the equations proposed for axial-shear failure of columns according to the Elwood-Moehle model (Elwood and Moehle, 2005) capture the displacements at which shear-dominated RC columns subject to unidirectional loading lose their axial load carrying capacity (Boys et al., 2008).
However, in many cases, and particularly when subjecting the column specimens to bidirectional loading, failure with loss of axial load capacity occurred at very low lateral drift levels: in the range of 1-1.5%.

Figure C5.2: Performance of poorly detailed and confined gravity columns designed according to NZS 3101:1982 code provisions (after Boys et al., 2008)

C5.3.3 Damage observations following the Canterbury earthquakes

C5.3.3.1 General

Tables C5.1 (pre mid-1970s RC buildings) and C5.2 (post mid-1970s RC buildings) provide a pictorial overview of the main structural deficiencies and observed damage of reinforced concrete buildings following the Canterbury earthquake sequence of 2010-2011.

For a more detailed overview of the seismic performance of RC buildings following the 4 September 2010 (Darfield Earthquake) and the 22 February 2011 (Lyttleton earthquake) events, refer to the NZSEE, 2010, 2011 and EERI/NZSEE 2014 Special Issues dedicated to the Canterbury Earthquake sequence (e. g. Kam et al., 2010, 2011; Fleischman et al., 2014; Sritharan et al., 2014 and Bech et al., 2014).

Note:

As the mid-1970s threshold cannot be taken as a rule to define earthquake risk buildings or earthquake prone buildings, it can be also argued that post mid-1970 concrete buildings are not expected to suddenly have superior seismic performance. In fact, research carried out under the FRST-funded ‘Retrofit Solutions’ project in New Zealand has confirmed that typical weaknesses of pre-1970s buildings were consistently adopted for several years subsequently (Pampanin et al., 2006-2010; Ingham et al., 2006).

For example, the issue of potentially inadequate transverse reinforcement observed in columns constructed since the 1960s was not completely addressed with the provisions of NZS 3101:1982, so that many buildings designed and constructed prior to the 1995 standards can be expected to have inadequate levels of confinement in their columns (potential SW) when compared to current standards. When confinement is low, loss of cover concrete combined with buckling of the longitudinal bars could occur, particularly in the lap-spliced regions, leading to unexpected failure.
Moreover, recent focus on displacement incompatibility issues between lateral load resisting systems (i.e. walls or floors) and floor systems has shown potential SWs. Inadequate structural details could favour local damage and failure mechanisms due to beam elongation and vertical displacement incompatibilities (refer to Section C5.5.6).

Irregularities in plan and elevation leading to torsionally-prone response, concentrated failure mechanisms, and/or ratcheting response have also been found as recurrent issues in post mid-1970 buildings.

Notwithstanding these comments, modern design philosophies were also being incorporated in buildings from the late 1960s as discussed in Section C5.2.3.

C5.3.3.2 Non-ductile concrete columns

The Canterbury earthquakes of 4 September 2010 and 22 February 2011 provided dramatic confirmation of the potentially high vulnerability of non-ductile gravity columns.

Figure C5.3 shows the example of two internal columns belonging to a parking structure (where the seismic resisting system consisted of steel K-braces in both directions) that was extensively damaged after the earthquake on 4 September 2010. The loss of axial load capacity due to lack of lateral drift capacity required immediate and urgent propping.

The Canterbury Earthquakes Royal Commission (CERC) report into the collapse of the CTV building in the 22 February 2011 Lyttleton earthquake found that the lack of ductile detailing in the gravity columns was likely to have been a contributing factor to the collapse of the building. The CTV building was designed in 1986 and had a six-storey reinforced concrete ‘shear wall protected’ gravity load system.

![Damage observed after the Canterbury earthquake on 4 September 2010](image)

![Experimental tests carried out years before on typical pre-1995 gravity-load columns subjected to bidirectional cyclic loading](image)

Figure C5.3: Severe damage with loss of vertical load-bearing capacity in columns with inadequate transverse reinforcement as part of the “gravity-load systems” due to displacement compatibility with the lateral load resisting systems (Kam and Pampanin, 2012)
Table C5.1: Typical/expected structural deficiencies and observed damage/failure mechanism in pre mid-1970s Canterbury RC buildings

<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams</td>
<td>Poor confinement details and transverse reinforcement in beams</td>
<td>Flexural plastic hinge in beams, often characterised by single crack opening (refer to photo below) - especially when plain round bars adopted.</td>
</tr>
<tr>
<td></td>
<td>Structural drawings of beam reinforcement and confinement details. Often the stirrups were ‘opened’ with a 90 degree angle instead of the more modern 135 degrees.</td>
<td>This would lead to higher deformability (fixed end rotation), lower moment capacity at a given drift demand and possibly excessive strain demand in the reinforcing steel bars. Also due to the poor confinement and transverse reinforcement details, higher level of demand could lead to premature compression-shear damage and failure in the plastic hinge region.</td>
</tr>
</tbody>
</table>

Structural drawings of beam reinforcement and confinement details.
<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inadequate anchorage of beam bars into the joint (refer to Section C5.5.4 – Beam-column joints)</td>
<td>(Refer to Joint section)</td>
</tr>
<tr>
<td></td>
<td>Inadequate splice detailing (short development length, $L_d$, well below 40 diameters)</td>
<td><img src="image1.jpg" alt="Photo" /> Observed lap-splice failure in beams due to limited splice length. Lapping was probably done at expected point of contraflexure due to gravity loading, without considering seismic effects.</td>
</tr>
<tr>
<td></td>
<td>Use of plain round (smooth) bars</td>
<td><img src="image2.jpg" alt="Photo" /> Development of single crack instead of a wider plastic hinge region. Concentration of strain and stresses in the reinforcing bars with possible premature failure in tension. Bond degradation and slip with reduced flexural capacity and energy dissipation (pinched hysteresis loop).</td>
</tr>
</tbody>
</table>
### Component or global structure

<table>
<thead>
<tr>
<th>Beam-column joints</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Lack or total absence of horizontal and/or vertical transverse reinforcement in the joint panel zone.</td>
<td>Shear damage/failure in joint area with potential loss of gravity load bearing capacity in column</td>
</tr>
</tbody>
</table>

**Figure:** Schematic illustrations of joint traverse reinforcement in pre-1970s buildings related to column stirrups and design assumptions:

- (a)-(b) Joint neglected in design or considered as a construction joint
- (c)-(d)-(e) Joints treated as part of column, therefore quantity of joint stirrups depended on column stirrup spacing and beam depth

**Figure and Photo:** Structural drawing of joint reinforcing details and observed shear failure of exterior joints. (It is worth noting that the failure in this case was due to a combination of lateral loading and vertical settlement due to failure of a foundation beam.)
<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inadequate anchorage of beam longitudinal bars into the joint</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lack of reliable joint shear transfer mechanism beyond diagonal cracking</td>
<td></td>
</tr>
</tbody>
</table>

Figure: Alternative structural detailing of non-ductile beam-column joint:
(a) 180° hooks (typical of plain round bars)
(b) beam bars bent into the joint with 90° inward bends
(c) beam bars bent out with 90° outwards bends
(d) top beam bars bent in at 90°, bottom bars stop short with no anchorage hook or bend
(e) top beam bars bent in at 90° bottom bars with hook anchorage (typically of plain round bars), and
(f) U-shaped bar splice into the joint core.
<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>Inadequate confinement detailing in the plastic hinge region:</td>
<td>Shear failure of the column at the plastic hinge</td>
</tr>
<tr>
<td></td>
<td>• not all of the bars of the longitudinal reinforcement are confined with stirrups</td>
<td>Buckling of the longitudinal reinforcement at the plastic hinge</td>
</tr>
<tr>
<td></td>
<td>• inadequate spacing for anti-buckling.</td>
<td>Photo: Example of shear failure and bucking of column in plastic hinge</td>
</tr>
</tbody>
</table>

Figure: Structural drawings of column confinement details

Photo: Example of shear failure and bucking of column in plastic hinge region
<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inadequate lap-splice details</td>
<td>Potential for weak-column/strong-beam mechanism due to significant decrease in the flexural capacity of the plastic hinge</td>
</tr>
<tr>
<td></td>
<td>Inadequate shear reinforcement</td>
<td>Potential shear failure</td>
</tr>
</tbody>
</table>

Figure: Structural drawing showing poor shear reinforcement details and lap splices

Photo: Shear failure of the columns due to short-column phenomenon
Short (captive) columns effects – effective shortening of the clear shear span of the columns due to presence of masonry or concrete infills, heavy spandrel beams or stiff non-structural facades

Shear failure of columns

Photo: Short column effect and shear failure due to presence of masonry infills

Photo: Short column effect due to presence of spandrel elements (bottom)
<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Walls</td>
<td>Inadequate longitudinal reinforcement ratio</td>
<td>Opening of single crack in the plastic hinge region, with concentration of strain demand and potential tensile failure of longitudinal bars</td>
</tr>
</tbody>
</table>

**Figure:** Structural drawing of a thin and singly reinforced wall

**Photo:** Tensile failure of longitudinal rebars hidden behind a single and small (residual) crack
<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inadequate confinement and shear reinforcement in walls</td>
<td>Crushing and buckling failure in the boundary regions</td>
</tr>
<tr>
<td></td>
<td><img src="image_url" alt="Figure: Structural drawing of confinement and shear reinforcement details in a wall" /></td>
<td><img src="image_url" alt="Photo: Wall failure due to buckled longitudinal reinforcements" /></td>
</tr>
<tr>
<td></td>
<td></td>
<td><img src="image_url" alt="Photo: Combination of buckling, single crack opening and shear sliding due to inadequate detailing" /></td>
</tr>
<tr>
<td>Component or global structure</td>
<td>Typical deficiency</td>
<td>Observed damage</td>
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<tr>
<td>-------------------------------</td>
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<td>-----------------</td>
</tr>
<tr>
<td>Inadequate lap-splice detailing</td>
<td>Photo: Crushing of end connection in boundary regions</td>
<td></td>
</tr>
<tr>
<td>Excessive wall slenderness ratio (wall height-to-thickness ratio)</td>
<td>Out-of-plane (lateral) instability Refer to example of associated observed damage in the following table (related to post mid-1970s walls)</td>
<td></td>
</tr>
<tr>
<td>Component or global structure</td>
<td>Typical deficiency</td>
<td>Observed damage</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>--------------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>Global structure</td>
<td>Lack of capacity design: weak-column, strong beam mechanism, soft-storey prone</td>
<td>Severe damages to columns or joints, which can lead to global brittle failure mechanism</td>
</tr>
</tbody>
</table>

Figure: Structural drawings of weak-column, strong beam mechanisms

Photos: Severe shear damage and failure in columns
### Component or Global Structure

<table>
<thead>
<tr>
<th>Component or Global Structure</th>
<th>Typical Deficiency</th>
<th>Observed Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>Lap-splicing with not sufficient length and confinement. More often away from the plastic hinge region.</td>
<td>Damage due to the compromised continuity of the element, loss of moment-capacity, potential soft-storey mechanism</td>
</tr>
</tbody>
</table>

**Figure:** Structural drawings showing inadequate lap-splicing

![Figure: Structural drawings showing inadequate lap-splicing](image1)

![Figure: Structural drawings showing inadequate lap-splicing](image2)
C5: Concrete Buildings

<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inadequate confinement at the plastic hinge region of columns with high axial load ratio</td>
<td>Shear-axial failure of columns</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Figure: Structural drawings of column confinement details</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Photo: Compression-shear failure in columns</td>
</tr>
<tr>
<td></td>
<td>Inadequate transverse reinforcement in circular columns to resist torsion</td>
<td>Torsional cracks</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Figure: Structural drawings showing transverse reinforcement details in circular column</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Photo: Torsional cracking of column</td>
</tr>
<tr>
<td>Component or global structure</td>
<td>Typical deficiency</td>
<td>Observed damage</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>--------------------------------------------------------------------------------------------------------</td>
<td>------------------------------------------------------------------</td>
</tr>
<tr>
<td>Walls</td>
<td>Inadequate confinement in boundary elements as well as core area</td>
<td>Crushing, spalling of concrete; bar buckling; out-of-plane failure</td>
</tr>
</tbody>
</table>

**Figure: Structural drawings of wall reinforcement and confinement details**

**Figure: Structural drawings of confinement details at wall corner and boundary element**

**Photos: Spalling of concrete at wall end, and buckling failure**

**Photos: Shear failure at ground floor wall**
Table C5.2: Typical/expected structural deficiencies and observed damage/failure mechanism in post mid-1970s Canterbury RC buildings

<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor/diaphragm</td>
<td>Beam elongation effects and lack of seating in precast floor diaphragms</td>
<td>Tearing/damage to diaphragm and potential loss of seating</td>
</tr>
<tr>
<td></td>
<td><img src="image" alt="Diagram of diaphragm damage" /></td>
<td>![Photos]</td>
</tr>
<tr>
<td></td>
<td><strong>Photos:</strong> Damage in the diaphragm due to beam elongation; potential unseating of floor units.</td>
<td></td>
</tr>
<tr>
<td>Non-ductile columns</td>
<td>Inadequate structural detailing to provide required ductility</td>
<td>Lack of capacity to sustain the imposed displacement-drift compatibly</td>
</tr>
<tr>
<td></td>
<td>Inadequate confinement and shear reinforcement, poor lap splices, excessive cover concrete</td>
<td>with the 3D response of the system</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Loss of gravity load bearing capacity at earlier level of interstorey drift</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Potential catastrophic collapse of the whole building</td>
</tr>
</tbody>
</table>
Photo: Example of details of pre-1995 non-ductile (secondary) columns. Large cover concrete, inadequate stirrups spacing.

Photos: Shear failure of pre-1995 non-ductile column details
**Walls**

<table>
<thead>
<tr>
<th>Flanged or irregular shaped walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure: Quasi-symmetric configuration of flanged-walls, yet leading to asymmetric response and inelastic torsion</td>
</tr>
</tbody>
</table>

| Local lateral instability and concentration of damage in compression region |
| Photos: Crushing of well confined boundary regions and lateral instability |
### Component or global structure

<table>
<thead>
<tr>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Under-designed boundary region, lack of ties in the web, inadequate design against bidirectional loading, including out-of-plane shear</td>
<td>Photo: Out-of-plane shear-buckling failure of shear wall</td>
</tr>
</tbody>
</table>

Figure: Example of actual details (top) of a 1980s shear walls and equivalent redesign according to latest NZS 3101:2006 design

### Global structure

<table>
<thead>
<tr>
<th>Plan irregularity</th>
<th>Damage due to torsional effect to components</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure: Irregular plan</td>
<td>Photos: Torsional cracks on columns</td>
</tr>
</tbody>
</table>

Photos: Torsional cracks on columns
### Component or global structure

<table>
<thead>
<tr>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;figure&gt;Plan irregularity&lt;/figure&gt;</td>
<td><img src="image" alt="Photo: Complete progressive collapse of the building as a result of a combination of a number of structural deficiencies including plan irregularity, non-ductile columns, weak diaphragm-to-lateral resisting system connection, etc." /></td>
</tr>
</tbody>
</table>

- Secondary Non-Ductile Columns
- Core Wall
- Coupled Wall

Photo: Complete progressive collapse of the building as a result of a combination of a number of structural deficiencies including plan irregularity, non-ductile columns, weak diaphragm-to-lateral resisting system connection, etc.
<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical irregularity</td>
<td></td>
<td>Photos: Vertical irregularity resulting in: (a) Severe basement columns shear-axial failure; (b) Transfer beam damage and repair; (c) and (d) Ground floor transfer slab and basement wall damage</td>
</tr>
</tbody>
</table>

Figure: Schematic plan of an 11-storey building with plan and vertical irregularity

Photos: (a) Severe basement columns shear-axial failure; (b) Transfer beam damage and repair; (c) and (d) Ground floor transfer slab and basement wall damage
### Global structure

<table>
<thead>
<tr>
<th>Vertical irregularity and set backs</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.jpg" alt="Photo: Vertical irregularity: set back" /></td>
</tr>
</tbody>
</table>

- Photo: Axial compression failure of ground floor column at the boundary of the setback. Transverse reinforcement: R6 spirals @ ~300-400 mm
<table>
<thead>
<tr>
<th>Component or global structure</th>
<th>Typical deficiency</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Photo: Captive column failure at building set-back level</td>
</tr>
</tbody>
</table>
### Component or Global Structure

<table>
<thead>
<tr>
<th>Global Structure</th>
<th>Typical Deficiency</th>
<th>Observed Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical irregularity and set-backs</td>
<td>Asymmetric behaviour leading to ratcheting response, concentration of damage in gravity load-bearing elements; e.g. base wall at the boundary with the setback and columns under transfer beam</td>
</tr>
</tbody>
</table>

Photo: Multi-storey building built mid-1980s with vertical irregularity due to first floor set-back and number of floors hanged on a transverse beam.

Photo: Axial-shear failure of columns under transverse beam due to ratcheting response.
C5.4 **Material Properties and Testing**

**C5.4.1 General**

For reinforced concrete structures, key material-related data for the assessment include:
- concrete strength (its probable strain capacity being indirectly derived/assumed)
- steel yield strength, probable tensile strength, probable strain capacity and the expected variation in its properties.

Information on the mechanical properties of concrete and steel reinforcing can be sourced from:
- the construction drawings, and/or
- the original design specifications, and/or
- original test reports, and/or
- in-situ testing.

As a starting point, and in the absence of further direct information, default values for the mechanical properties of the reinforcing steel may be assumed in accordance with the relevant standards at the time of construction. The following sections provide reference values and summaries of the evolution of concrete and steel reinforcing material standards. More details on the historical material properties specifications and design requirements in New Zealand can be found in Appendix C5C or C5D.

**Note:**

Proper integration of different sources will be required to improve the level of knowledge and confidence in structural material properties and, therefore, in the assessment outputs.

Any in-situ testing – whether limited, extensive or comprehensive – should be specifically targeted to improve confidence in the assessment result; e.g. as part of the evaluation of the hierarchy of strength between connected elements or within the same element.

The extent of in-situ testing should be based on an assessment of the tangible benefits that will be obtained. It will not be practical to test all materials and in all location, but the investigation can be restricted to the elements within the most critical mechanisms.

To address this issue it is considered reasonable to adopt the general material strengths as outlined below after first making an assessment on general material quality (particularly in relation to the concrete work). If there is no indication of the targeted (specified) material strengths in the construction documentation, a suitably scoped investigation program may be required to determine the concrete and reinforcing steel strengths that were likely to have been specified and targeted. An example of guidance on the number of in-situ tests needed to get a statistically meaningful result can be found in Sezen et al., 2011.

Refer to Appendix C5C for a detailed list of alternative destructive and non-destructive techniques for gathering further information on concrete and reinforcing steel material properties.
Note:
For some mechanisms it may be necessary to consider the potential variation in material strengths so that the hierarchy of strength and sequence of events can be reasonably assessed and allowed for. Provision has been allowed for some of this variation in element capacity calculations (e.g. shear). Otherwise, this may need to be specifically accounted for if full benefit from the formation of a particular local or global mechanism is to be relied on.

Use of probable and overstrength element capacities as outlined in these guidelines is considered to provide the required level of confidence that a mechanism will be able to develop with the required hierarchy.

C5.4.2 Concrete

C5.4.2.1 General

Regardless of the information provided on the drawings, the actual properties of concrete used in the building might vary significantly. This can be due to factors such as:

- construction practice at the time the building was constructed; e.g. poor placement and compaction, addition of water for workability
- the fact that the concrete may have been subject to less stringent quality control tests on site, and
- concrete aging.

Appendix C5A summarises the evolution of concrete property requirements and design specifications in New Zealand. Appendix C5B summarises the tests used for quality control of concrete as contained in the New Zealand standard for specification for concrete production, NZS 3104, from 1983 to the present day.

Notwithstanding the potential inherent variability in concrete properties, which will be impossible to precisely determine (even with extensive investigation), it is intended that a seismic assessment be based on reasonably established generic concrete properties as outlined below.

C5.4.2.2 Probable compressive strength of concrete

In the absence of (as well as prior) more information from detailed on-site testing, the probable compressive strength of concrete, \( f'_{c,\text{prob}} \), may be taken as the nominal 28-day compressive strength of the concrete specified for construction, \( f'_{c} \), (lower bound compressive strength) factored by 1.5 – which accounts for, amongst other factors, the ratio between probable and fifth-percentile values as well as aging effects.

Table C5.3 presents suggested probable values for the compression strength of concrete depending on the age and minimum compressive strength specified (lower-bound) at different periods based on NZS 3101:2006.
Table C5.3. Default lower-bound concrete compressive strengths, as specified in various New Zealand concrete standards, and suggested probable compressive strengths

<table>
<thead>
<tr>
<th>Period</th>
<th>Lower-bound compressive strength (MPa)</th>
<th>Suggested f probable compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f'_{ck}$</td>
<td>$f'_{cm}$</td>
</tr>
<tr>
<td>1970-1981</td>
<td>17.2</td>
<td>25</td>
</tr>
<tr>
<td>1982-1994</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>1995-2005</td>
<td>17.5</td>
<td>25</td>
</tr>
<tr>
<td>2006-present</td>
<td>25</td>
<td>35</td>
</tr>
</tbody>
</table>

**Note:**
The actual compressive strength of old concrete is likely to exceed the specified value as a result of conservative mix design, aging effect and the coarser cement particles that were used. Furthermore, probable strength values should be used for assessment, instead of fifth-percentile values (or lower bound of compression strength) typically adopted for design.

There is a lack of experimental in situ testing of New Zealand structures, and of buildings in particular, to allow the strength of aged concrete to be reliably determined.

As an indicative reference only, tests on the concrete of 30-year-old bridges in California consistently showed compressive strengths approximately twice the specified strength (Priestley, 1995). Concrete from the columns of the Thorndon overbridge in Wellington had a measured compressive strength of about 2.3 times the specified value of 27.5 MPa about 30 years after construction (Park, 1996).

Similarly, concrete from collapsed columns of the elevated Hanshin Expressway in Kobe, Japan after the January 1995 earthquake had a measured compressive strength of about 1.8 times the specified value of 27.5 MPa almost 30 years after construction (Park, 1996), (Presland, 1999).

Eurocode 2 Part 1, 2004 provides an expression to evaluate the aging factor as a function of the strength class of cement adopted. The aging factor tends almost asymptotically after 10-20 years to values in the range of 1.2-1.4 depending on the cement strength class.

This limited evidence, at least, would suggest that the use of a factor of 1.5 between the originally specified concrete strength (lower bound – fifth percentile) and the probable concrete strength can be considered a reasonable value.
C5.4.3 Reinforcing steel rebars

C5.4.3.1 General

The historical overview below should provide a useful basis for the expected mechanical characteristics of reinforcing steel if more specific information is not available from the building’s structural and construction drawings.

However, any reliance on this information should be supported, whenever practicable and as required, with in-situ investigation and testing on sample specimens to obtain a better estimation of the reinforcement’s probable yield strength; or at least to confirm the grade of reinforcement that was used.

C5.4.3.2 History of steel reinforcement in New Zealand

The first New Zealand standard to regulate the mechanical properties of steel bars for reinforcing concrete is likely to have been NZS 197:1949 (based on BS 785:1938) “ Rolled steel bars and hard drawn steel wire”. This standard only referred to plain round bars.

Before NZS 197:1949 (BS 785:1938), there was apparently no specific national standard to cover reinforcing steel. However, it can be reasonably assumed that steel reinforcement was regulated by BS 165:1929, which was the previous version of BS 785:1938 used in New Zealand from 1949.

Deformed bars were introduced in 1963 with NZSS 1693:1962 “Deformed steel bars ofstructural grade for Reinforced Concrete”. A 227 MPa (33,000 psi) yield stress steel bar was first introduced and then replaced in 1968 (Amendment 1 of NZSS 1693:1962) by a 275 MPa (40,000 psi yield stress steel bar).

Note:

It can therefore be assumed that plain round bars were used in concrete buildings at least until the mid-1960s. The required development length for plain round bars can be taken as not less than twice that for deformed bars specified in NZS 3101 (2006).

Also note that during cyclic loading the bond degradation for plain round bars is more significant than for deformed bars (Liu and Park, 1998 and 2001; Pampanin et al., 2002). Hence, old structures reinforced with plain round longitudinal bars will show a greater reduction in stiffness during cyclic loading. As a reference value, as part of quasi-static cyclic load tests of beam-column joint subassemblies reinforced by plain round longitudinal bars at the University of Canterbury, the measured lateral displacements were approximately twice those of similar assemblies reinforced by deformed longitudinal bars at similar stages of loading (Liu and Park, 1998 and 2001).

Often plain round bars were terminated with hooks to provide reliable development of the bars, but this was not always the case.

In 1964 another standard relating to deformed steel bars was issued: NZSS 1879:1964 “Hot rolled deformed bars of HY 60 (High yield 60,000 psi) for Reinforced Concrete”. This standard introduced a higher yield steel bar with a yield stress of about 414 MPa (60,000 psi). At this stage, there were three standards for steel reinforcing bars: one for plain round bars (NZS 197) and two for deformed bars (NZSS 1693 and NZSS 1879).
Note:

Chapman (1991) reports that site sampling and testing has found the structural grade reinforcement in New Zealand structures built during the 1930s to 1970s is likely to possess a lower characteristic yield strength (fifth percentile value) that is 15-20% greater than the specified value.

Reinforcing steel from the pile caps of the Thorndon overbridge in Wellington constructed in the 1960s had a measured mean yield strength of 318 MPa with a standard deviation of 19 MPa (Prestland, 1999).

In 1972 the old NZS 197 was replaced by a temporary standard NZS 3423P:1972 “Hot rolled plain round steel bars of structural grade for reinforced concrete” but this was only valid for a year. In 1973, all three standards (NZSS 1693:1962, NZSS 1879:1964 and NZS 3423P) were superseded by NZS 3402P:1973 “Hot rolled steel bars for the reinforcement of concrete” which regulated both plain round and deformed bars.

Metric units for steel bars were slowly introduced in 1974 and became the only units used by steel manufacturers from 1976 onwards. Steel grades used at that time were Grade 275 and Grade 380.

In 1989, NZS 3402P was superseded by NZS 3402:1989. This replaced Grades 275 and 380 with new grades, 300 and 430.

Note:

It is common practice to assume a Grade 300 steel for a modern (post-1980s) existing building. However, as shown in this section and in the table below, the evolution of steel properties in the past decade has been quite significant. Therefore, confirming this assumption by reference to the drawings as well as a minimum level of on-site sampling and testing is recommended.

In 2001, the current version of the standard for reinforcing steel, AS/NZS 4671:2001, was introduced. Steel grades proposed for New Zealand in this standard are Grade 300E (Earthquake ductility) and Grade 500E.

Table C5.4 summarises the evolution of these standards, while Tables C5.5 and C5.6 list available diameters for steel reinforcing bars. Also refer to Appendix C5D for a summary of the historical evolution of the mechanical properties of steel reinforcing over different time periods.
### Table C5.4: Evolution of reinforcing steel material standards in New Zealand

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>NZS 197:1949 (BS 785:1938) Rolled steel bars and drawn steel wire for concrete reinforcement (Yield stress varied with diameter, minimum value was 227 MPa, Refer to Appendix C5D.1)</td>
<td>NZS 3423P:1972 Hot rolled plain round steel bars of structural grade for reinforced concrete &quot;Grade&quot; 40,000 psi (275 MPa)</td>
<td>NZS 3402P:1973 Hot rolled steel bars for the reinforcement of concrete Grade 275 MPa Grade 380 MPa</td>
<td>NZS 3402:1989 Steel bars for the reinforcement of concrete Grade 300 MPa Grade 430 MPa</td>
<td>AS/NZS 4671:2001 Steel reinforcing material Grade 300 MPa Grade 500 MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NZSS 1693:1962 Deformed steel bars of structural grade for reinforced concrete &quot;Grade&quot; 33000 psi (227 MPa)</td>
<td>NZS 1879:1964 Hot rolled deformed bars of HY 60 (High Yield 60,000 psi) for reinforced concrete Grade&quot; 60,000 psi (415 MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table C5.5: Available diameters of steel reinforcement bars – before the mid-1970s

<table>
<thead>
<tr>
<th>Bar designation</th>
<th>d inch (mm)</th>
<th>Bar designation</th>
<th>d inch (mm)</th>
<th>Bar designation</th>
<th>d inch (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>1/2 (12.7)</td>
<td>4</td>
<td>1/2 (12.7)</td>
<td>4</td>
<td>1/2 (12.7)</td>
</tr>
<tr>
<td>5</td>
<td>5/8 (15.875)</td>
<td>5</td>
<td>5/8 (15.875)</td>
<td>5</td>
<td>5/8 (15.875)</td>
</tr>
<tr>
<td>6</td>
<td>3/4 (19.05)</td>
<td>6</td>
<td>3/4 (19.05)</td>
<td>6</td>
<td>3/4 (19.05)</td>
</tr>
<tr>
<td>7</td>
<td>7/8 (22.225)</td>
<td>7</td>
<td>7/8 (22.225)</td>
<td>7</td>
<td>7/8 (22.225)</td>
</tr>
<tr>
<td>8</td>
<td>1.000 (25.4)</td>
<td>8</td>
<td>1.000 (25.4)</td>
<td>8</td>
<td>1.000 (25.4)</td>
</tr>
<tr>
<td>9</td>
<td>1 1/8 (28.575)</td>
<td>9</td>
<td>1 1/8 (28.575)</td>
<td>9</td>
<td>1 1/8 (28.575)</td>
</tr>
<tr>
<td>10</td>
<td>1 1/4 (31. 75)</td>
<td>10</td>
<td>1 1/4 (31. 75)</td>
<td>10</td>
<td>1 1/4 (31. 75)</td>
</tr>
<tr>
<td>11</td>
<td>1 3/8 (34.925)</td>
<td>11</td>
<td>1 3/8 (34.925)</td>
<td>11</td>
<td>1 3/8 (34.925)</td>
</tr>
<tr>
<td>12*</td>
<td>1 1/2*(38.1)</td>
<td>12*</td>
<td>1 1/2*(38.1)</td>
<td>12*</td>
<td>1 1/2*(38.1)</td>
</tr>
</tbody>
</table>

**Note:**
- * Introduced in 1970
Table C5.6: Available diameters of steel reinforcement bars – from the mid-1970s onward

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bar designation d (inch)</td>
<td>d (mm)</td>
<td>Bar designation d (inch)</td>
<td>d (mm)</td>
</tr>
<tr>
<td>R10 D10</td>
<td>-</td>
<td>10</td>
<td>R10 D10</td>
</tr>
<tr>
<td>R13 D13</td>
<td>⅝</td>
<td>12.7</td>
<td>R12 D12</td>
</tr>
<tr>
<td>R16 D16</td>
<td>-</td>
<td>16</td>
<td>R16 D16</td>
</tr>
<tr>
<td>R20 D20</td>
<td>-</td>
<td>20</td>
<td>R20 D20</td>
</tr>
<tr>
<td>R22 D22</td>
<td>7/8</td>
<td>22.23</td>
<td>R24 D24</td>
</tr>
<tr>
<td>R32 D32</td>
<td>-</td>
<td>32</td>
<td>R40 D40</td>
</tr>
<tr>
<td>R38 D38</td>
<td>1 ⅜</td>
<td>38.1</td>
<td>R40 D40</td>
</tr>
</tbody>
</table>

C5.4.3.3 Probable yield strength of reinforcing steel

The probable yield strength of the reinforcing steel may be taken as the mean of the upper characteristic (95th percentile value) and the lower characteristic (fifth percentile value) yield strength.

Note:
The ratio between the upper and lower characteristic yield strengths will typically be in the range of 1.17 to 1.3 depending on source and age. Refer to Appendix C5D.1 for indicative values. Hence, based on the lower end of the expected range, the probable yield strength of the reinforcing steel may be taken as 1.08 times the lower characteristic yield strength.

C5.4.4 Cold wire mesh

Steel wire for concrete reinforcement was originally regulated in New Zealand by the first local steel code NZS 197:1949 (BS 785:1938). The tensile strength limits were between 37 ton/in² (510 MPa) and 42 ton/in² (580 MPa). The elongation limit was 7.5% measured over a gauge length of 8 times the diameter. This standard remained valid until 1972.

In 1972, NZS 3421:1972 and NZS 3422:1972 replaced the old standard. The first of these provided specifications for hard drawn steel wire; the second, for welded fabric hard drawn steel wire. Hard drawn steel wires were normally available in diameters not greater than 0.1 inches (12.7 mm) and not less than 0.08 inches (2.0 mm). The minimum 0.2 percent proof stress limit was 70,000 lbf/in² (483 MPa) while the minimum tensile strength was 83,000 lbf/in² (572 MPa). The mechanical property limits of welded fabric of drawn steel wires were similar to the ones specified for hard drawn steel wires. A maximum tensile strength limit was introduced equal to 124,000 lbf/in² (855 MPa) for diameters up to and including 0.128 in (3.25 mm) and 112,000 lbf/in² (772 MPa) for diameters over 0.128 in.

In 1975 NZS 3421:1972 and NZS 3422:1972 were superseded by the metric units versions NZS 3421:1975 Hard drawn steel wire for concrete reinforcement (metric units) and NZS 3422:1975 Welded fabric of drawn steel wire for concrete reinforcement (metric units). The first was applied to plain and deformed wires while the second only to plain ones.
The available diameters ranged between 2.5 mm and 8 mm. The mechanical property limits were similar to those prescribed in the 1972 standards: 485 MPa for minimum 0.2 percent prof stress; 575 MPa for minimum tensile strength and 855 MPa maximum tensile strength (for diameters up and including 3.15 mm) and 775 MPa (for diameters over 3.15 mm).

The current AS/NZS 4671:2001 (Steel reinforcing materials) replaced the old NZS 3421:1975 and NZS 3422:1975. This standard provides specifications for steel reinforcing bars and mesh. The steel grades are Grade 300E and Grade 500E. The commonly available mesh diameters are 6 mm, 7 mm, 8 mm and 9 mm for structural mesh and 4 mm and 5.3 mm for non-structural mesh. The most common mesh pitch size for is 200 by 200 mm for structural mesh and 150 by 150 mm for non-structural mesh.

<table>
<thead>
<tr>
<th>Year</th>
<th>1949</th>
<th>1972</th>
<th>1975</th>
<th>2001</th>
</tr>
</thead>
</table>

### C5.5 Element Probable Capacities

#### C5.5.1 General approach

This section sets out the procedure for evaluating the probable strength and deformation capacities of beams, columns, beam-column joints, walls and diaphragms.

The general approach taken to determine RC probable element capacities is to:

- evaluate the probable flexural strength and deformation capacity
- evaluate the probable shear strength and deformation capacity, and
- determine which mechanism is likely to occur first and at what level of deformation/displacement/drift.

The following Sections (C5.6, C5.7 and C5.8) describe how to:

- evaluate the hierarchy of strength between connected elements to determine the likely subassembly and system mechanism, and
- derive the global force-displacement curve associated with the expected mechanism for frame systems, walls and dual systems.

In order to evaluate the global capacity of the building, the capacities of individual structural systems should then be combined with those of other systems within the building, which could be also composed of other materials, in accordance with the guidance provided in Section C2 to evaluate the global capacity of the building.
C5.5.1.1 Key terms

The following key terms are used in the derivation of probable element capacities outlined in the following sections.

Nominal capacity

For reinforced concrete the nominal strength capacity, $S_n$, is the theoretical strength of a member section based on established theory, calculated using the section dimensions as detailed and the lower characteristic reinforcement yield strengths (fifth percentile values) and the specified nominal compressive strength of the concrete.

The nominal strength capacity gives a lower bound to the strength of the section and is the value typically used for design.

Similarly, for design, the nominal deformation capacity is determined in accordance with the concrete design standard NZS 3101:2006.

For assessment, the probable values as defined below shall be used.

Probable capacity

The probable strength capacity, $S_{prob}$, which is also referred to as expected strength capacity, is the theoretical strength of a member section based on established theory, calculated using the section dimensions as detailed and the probable (mean values) material strengths.

The probable or expected deformation capacity is determined as indicated in the following sections.

Overstrength

The overstrength capacity, $S_o$, takes into account factors that may contribute to an increase in strength such as: higher than specified strengths of the steel and concrete, steel strain hardening, confinement of concrete, and additional reinforcement placed for construction and otherwise unaccounted for in calculations.

For beams, the overstrength in flexure, when tension failure is controlling the ULS behaviour, is mainly due to the steel properties along with the slab flange effect and possibly the increase in axial load due to beam elongation. For current New Zealand manufactured reinforcing steel, an upper bound for the yield strength can be taken as the upper characteristic (95th percentile value).
A further 8% increase in steel stress due to strain hardening should be assumed (e.g. refer to Andriono and Park, 1986).

Hence, as a first approximation – i.e. as a quick check before more comprehensive calculations – and indicatively, the ratio of overstrength in flexure to:

- nominal flexural strength, $M_o/M_n$, can be taken as 1.25 (for both Grade 300 and Grade 430 steel) and 1.35 for Grade 500
- probable flexural strength, $M_o/M_{prob}$, can be taken as 1.16.

For columns, while adequate confinement can cause an increase in the concrete compressive strain and ultimate deformation capacity, the effect on the increase in flexural strength is limited. For poorly detailed and confined columns this enhancement in flexural strength is further limited, such that neglecting it would be on the conservative side.

**Strength reduction factor**

For the purposes of calculating the probable strength capacity no strength reduction factor $\phi$ should be used for either flexure or shear (i.e. $\phi=1.0$). Where considered necessary, a strength reduction factor to provide a safety margin against shear failure has been included in the derivation of the shear capacity equations.

**Bounds of flexural strength**

The lower and upper bounds of flexural strength are important when assessing moment resisting frames to determine the hierarchy of strength mechanism of post-elastic deformation. For beams and columns the lower bound of flexural strength can be taken as the probable strength, and the upper bound as the overstrength.

**Note:**

In such calculations it is important to account for the variation of axial load due to lateral sway mechanism (e.g. frame action) and/or due to displacement incompatibility issues (e.g. vertical restraint from floor during lifting up of wall or horizontal restraint to beams due to beam elongation effects).
C5.5.2 Beam capacity

C5.5.2.1 History of code-based reinforcement requirements for beams in New Zealand

If structural and/or construction drawings for the building are not available, it may be useful to refer to the New Zealand standards of the time. Appendix C5D.2 summarises the structural detail requirements for beams according to the NZS 3101:2006 standards from 1970 onwards (1970, 1982, 1995 and 2006). More information can be found in Cuevas et al. (2015).

Figure C5.4 illustrates the evolution of structural design requirements and detailing layout for beams according to the New Zealand concrete standard from the 1970s onwards.

Figure C5.4: Example of typical beam layouts according to different versions of NZS 3101:2006 (Cuevas et al., 2015)
C5.5.2.2 Probable flexural strength of beams

General

The probable flexural strength of members should be calculated using the probable material strengths and, with some care, the standard theory for flexural strength (Park and Paulay, 1975).

It is worth recalling that the basic theory for RC section flexural strength (refer to Figure C5.5) relies upon key assumptions such as:

- plane section remain plane (Hooke 1678, also known as Bernoulli-Navier theory), and
- fully bonded conditions between steel and concrete (i.e. no or negligible bond slip).

![Figure C5.5: Schematic of section analysis for RC flexural theory with assumptions on plane sections and fully bonded conditions](image)

While these assumptions are generally valid for modern and relatively well designed members, issues can arise when dealing with older construction detailing; in particular, inadequate anchorage/development length and/or use of plain round bars.

In these cases, the flexural capacity as well as the probable curvature and ductility capacity of the beams and columns can be reduced. In turn, this can affect the hierarchy of strength within a beam-column joint connection/subassembly as discussed in subsequent sections.

Note:

The plastic hinges in the beams normally occur at or near the beam ends in seismically dominated frames (whilst in gravity-dominated frames these could occur away from the column interface). Therefore, the longitudinal beam reinforcement is at or near the yield strength at the column faces.

This can result in high bond stresses along beam bars which pass through an interior joint core, since a beam bar can be close to yield in compression at one column face and at yield in tension at the other column face (refer to Figure C5.6). During severe cyclic loading caused by earthquake actions, bond deterioration may occur in the joint. If the bond deterioration is significant, the bar tension will penetrate through the joint core, and the bar tensile force will be anchored in the beam on the far side of the joint.

This means that the compression steel will actually be in tension. As a result, the probable flexural strength and the probable curvature capacity of the beam will be reduced.
Hakuto et al. (1999) have analysed doubly reinforced beam sections at the face of columns of a typical building frame constructed in New Zealand in the late 1950s. The effect of stress levels in the “compression” reinforcement on the moment capacity of the beam was not found to be significant. When the bond had deteriorated to the extent that the “compression” reinforcement was at the yield strength in tension, the decrease in moment capacity was up to 10% for positive moment and up to 5% for negative moment compared with beams with perfect bond along the beam bars (Hakuto et al., 1999).

To conclude, based on this evidence and in order to provide a simplified procedure, the effect of bar slip on flexural strength of beams could be neglected in the assessment.

Similarly, for the first approximation the reduced level of ductility demand can be calculated by ignoring the compression reinforcement (in case a tension failure mechanism is expected).

Note that the bond-slip could actually introduce additional sources of deformability, increasing the deformation demand in the structural system.

The flexural strength of columns within a beam-column joint is similarly affected due to bond-slip of the longitudinal vertical reinforcement, as discussed in Section C5.5.3.

In general terms, consideration of the upper and lower bounds of flexural strength of beams and columns is important when assessing the behaviour of moment resisting frames to determine the likely hierarchy of strength and global mechanism, and therefore whether plastic hinging will occur in the beams or columns.

**Slab and transverse beam contributions to negative flexural capacity of beams**

When calculating the probable flexural capacity of beams in negative moment regions it is important to account for the potential “flange-effect” contribution from the slab reinforcement (refer to Figure C5.7). This is particularly important when cast-in-place floor...
slabs (which are integrally built with the beams) are used. However, it should not be underestimated when precast floors with topping and starter bars are used.

Experimental evidence has also revealed the influence of the transverse beam torsion resistance on the magnitude of the effective width due to flange effect, $b_{eff}$, in exterior beam-column joints of cast-in-place two-way frames (Durrani and Zerbe, 1987; Di Franco et al., 1995).

A higher-than expected strength of the beam could modify the hierarchy of strength in a beam-column joint, possibly resulting into an increased risk of a column-sway mechanism when compared to a more desirable beam-sway mechanism.

As a first approximation the slab can be assumed to provide a 50% increase in the beam negative probable moment capacity and corresponding overstrength capacity, as shown in Figure C5.7. However, experimental research has shown that the presence of a slab and transverse beam can increase the negative flexural strength of a beam by up to 1.7 to 2 times (Durrani and Zerbe, 1987; Ehsani and Wight, 1985; Di Franco, Shin and La Fave, 2004).

Therefore, it is recommended that the overstrength capacity should be more properly evaluated in cases where the hierarchy of hinge formation within the mechanism is important to the assessment result.

In addition to increasing the flexural capacity, the slab reinforcement reduces the ultimate ductility of curvature of a beam section.

**Figure C5.7:** (a) Schematic monolithic one-way floor slab with beams (b) T-beam in double-bending (c) X-sections of T-beam showing different tension and compression zones (MacGregor, 1997)

**Note:**

The actual contributions of slab reinforcement to the negative moment flexural strength of a beam is dependent on: (1) the type of floor system, (2) the boundary conditions of the slab, (3) the level of imposed deformation on the beam-slab section, (4) the torsional resistance of transverse beams, if any, and (5) the quality of the anchorage of the reinforcing bars to develop full tensile strength.
In absence of further analysis, the recommendations provided by a new building standard (such as NZS 3101:2006) to evaluate the width of the slab contributing, with its reinforcement, to the flexural capacity under negative moments of T and L beams built integrally with the slab can be taken as a reference.

In poorly detailed beam-column joints where the joint and column are weaker than the beam-slab section, an effective width of the slab $b_{\text{eff}} = 2.2h_b$ (which includes the width of the beam) can be also used as a reference, based on the experimental research conducted by Kam et al., 2010.

To account for the torsional effects of a transverse beam, these guidelines recommend an effective width $b_{\text{eff}} = b_c + 2h_t$, where $b_c$ is the width of the column and $h_t$ is the height of the transverse beam or spandrel.

### C5.5.2.3 Flexural-shear interaction

The moment (or lateral force) curvature (or rotation/displacement) capacity curve of the element (beam, column, wall) calculated assuming flexural behaviour As discussed above (refer to Section C5.5.2.2) – i.e. with no shear interaction – can intersect the shear strength capacity curve including degradation due to ductility demand in the plastic hinge as shown in Figure C5.8.

This will limit the flexural deformation capacity of the plastic hinge to the value at the point of intersection and/or make the mechanism shear dominated at that hinge location for larger plastic hinge deformations.

![Figure C5.8: Flexural-shear interaction on a component (beam or column) capacity curve](image)

### C5.5.2.4 Probable shear strength of beams

The probable shear strength capacity of beams with rectangular stirrups or hoops can be taken as:

$$V_p = 0.85 (V_c + V_s) \quad \cdots \text{C5.1}$$

where $V_c$ and $V_s$ are the shear contributions provided by the concrete mechanism and steel shear reinforcement respectively.
In more detail:

- the shear contribution from the concrete, $V_c$, can be evaluated as:

$$V_c = \alpha \beta \gamma \sqrt{f'_c (0.8A_g)}$$  \[C5.2\]

where:

$$1 \leq \alpha = 3 - \frac{M}{V_D} \leq 1.5$$

$$\beta = 0.5 + 20 \rho_t \leq 1$$

$$\gamma = \text{shear strength degradation factor in the plastic hinge region due to ductility demand as defined in Figure C5.9}$$

$$A_g = b_w d = \text{gross area of the beam}$$

$$b_w = \text{width of beam web}$$

$$d = \text{effective depth of beam}$$

$$f'_c = \text{probable concrete compressive strength}$$

$$M/V = \text{ratio of moment to shear at the section}$$

$$D = \text{total section depth}$$

$$\rho_t = \text{longitudinal tensile reinforcement ratio, i.e. area of longitudinal beam tension (only) reinforcement divided by the cross-sectional area.}$$

- The shear contribution from the steel shear reinforcement, $V_s$, is evaluated assuming that the critical diagonal tension crack is inclined at 45° to the longitudinal axis of the column.

$$V_s = \frac{A_v f_{yt} d}{s}$$  \[C5.3\]

where:

$$A_v = \text{total effective area of hoops and cross ties in the direction of the shear force at spacing } s$$

$$f_{yt} = \text{expected yield strength of the transverse reinforcement}$$

$$d = \text{effective depth of the beam.}$$

Figure C5.9: Shear strength degradation factor, $\gamma$, based on curvature ductility within the plastic hinge Priestley et al. (2007) - based on test results from Hakuto et al. (1995); Priestley, (1995) and Priestley et al. (1994)
**Note:**

Within the plastic hinge region the probable shear strength of beams degrades as the level of imposed curvature ductility $\phi/\phi_y$ increases as shown in Figure C5.9 as proposed by Priestley (1995) and Priestley et al. (2007).

This shear strength degradation is due to the reduction of the shear contribution by the concrete mechanisms and depends on the curvature ductility demand.

### C5.5.2.5 Probable deformation (curvature and rotation) capacity of beams

#### Yield curvature and rotation

The yield curvature of a beam can be evaluated using a section analysis.

For a beam the first yield curvature is given by:

$$
\phi_y = \frac{\varepsilon_y}{d - kd} \quad \text{...C5.4}
$$

where:

- $\varepsilon_y = \text{strain at first yield of the longitudinal tension reinforcement (} = \frac{f_y}{E_s}\) $
- $d = \text{effective depth of longitudinal tension reinforcement}$
- $kd = \text{neutral axis depth when tension steel reaches the strain at first yield, } \varepsilon_y$.

In principle, and particularly for multiple layers of reinforcement in beams (and more commonly for columns), $\phi_y$ should be defined using a bilinear approximation (refer to Figure C5.14).

Priestley and Kowalsky (2000) have shown that the yield curvature can be well approximated with dimensionless formulae with minimal variations due to the axial load and reinforcement ratio as follows.

For rectangular-section beams:

$$
\phi_y = \frac{2.0 \varepsilon_y}{h_b} \quad \text{...C5.5}
$$

For T-section beams:

$$
\phi_y = \frac{1.7 \varepsilon_y}{h_b} \quad \text{...C5.6}
$$

where:

- $h_b = \text{beam depth}$
Probable ultimate curvature and rotation capacity

The available curvature ductility factor at a plastic hinge is given by $\mu = \phi_u / \phi_y$ where $\phi_u$ is the available probable ultimate curvature and $\phi_y$ is the yield curvature (determined as above).

When evaluating the probable ultimate curvature and rotation capacity of a beam a material strain (limit) needs to be adopted that reflects the level of detailing (confinement) provided. Table C5.8 provides suggested strain-based values for the evaluation of ULS capacity of beams and columns.

The key limit states within a moment-curvature or force-displacement curve of a structural element (i.e. cracking, yielding, spalling and ultimate) are shown in Figure C5.10.

### Table C5.8: Concrete and steel strain limits corresponding to ultimate limit state (ULS)

<table>
<thead>
<tr>
<th>Limit states</th>
<th>Concrete</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_{cu}$</td>
<td>$0.004 + \frac{1.4\rho_s f_y \varepsilon_{su}}{f_{cc}} \leq 0.015$ (confined core)</td>
<td>$0.06 \leq 0.6 (\varepsilon_{su})$</td>
</tr>
</tbody>
</table>

![Figure C5.10: Key limit states in a moment-curvature and force-displacement curve](image)

**Note:**

In general terms, for assessment purposes, the ULS is not assumed to be reached when at the crushing or spalling in the cover concrete occur, with a value of compression strain in the extreme fiber $\varepsilon_c = 0.004$ (the typical approach used for ULS design of new elements), but rather when either:

(i) an overall strength reduction of more than 20% occurs or
(ii) the confined concrete-core reaches the confined concrete strain limit or
(iii) the steel reaches much higher level of strain (e.g. $\varepsilon_s = 0.06$).
These potential deformation resources of an existing beam element beyond crushing and spalling of the cover concrete can be appreciated in the moment-curvature example given in Figure C5.11.

On the other hand, especially in columns with high axial load ratio, poor confinement detailing and large cover concrete, the loss of cover concrete (resulting from or combined with buckling of the longitudinal rebars) can correspond to the onset of full collapse.

This confirms that it is critical to carefully evaluate the expected mechanisms in existing structural elements designed to older design provisions.

![Figure C5.11: Example of a moment-curvature for a flanged (T or L) beam and strain limits at damage control limit state and ultimate limit state](image)

The available probable ultimate curvature for a beam is given by the lesser of:

\[
\phi_u = \frac{\varepsilon_{cu}}{c_u} \quad \text{...C5.7}
\]

and:

\[
\phi_u = \frac{\varepsilon_{su}}{d-c_u} \quad \text{...C5.8}
\]

where:

- \( c_u \) = neutral axis depth at ULS
- \( \varepsilon_{cu} \) = the ultimate concrete compressive strain, at the extreme fiber of the section or of the confined core region, depending on the extent of confinement of the concrete (as defined in Table C5.8 and further explained below)
- \( \varepsilon_{su} \) = the maximum accepted strain of the reinforcing steel in tension (as defined in Table C5.8)
- \( d \) = effective depth of longitudinal tension reinforcement

For unconfined concrete \( \varepsilon_{cu} = \varepsilon_{sp} = 0.004 \) can be assumed (Priestley and Park, 1987).
“Unconfined” conditions are assumed to be present if at least one of the following applies:
- only corner bars restrained against buckling by a bend of transverse reinforcement, or
- hoop stirrup ends not bent back into the core (i.e. 90° hooks), or
- spacing of hoop or stirrup sets in the potential plastic hinge such that:

\[
\begin{align*}
  s &\geq d/2 \\
  \text{or} \\
  s &\geq 16d_b
\end{align*}
\]

where:
- \(d\) = effective depth of beam section
- \(d_b\) = diameter of longitudinal reinforcement

For confined concrete the ultimate probable concrete strain can be assumed from a modification (fib, Bulletin 25) of the expression proposed Mander et al. (1988).

\[
\varepsilon_{cu} = 0.004 + \frac{1.4\rho_v f_{yh}\varepsilon_{su}}{f'_{cc}} \leq 0.015 \quad \ldots \text{C5.9}
\]

where:
- \(\rho_v\) = volumetric ratio of transverse reinforcement
- \(f_{yh}\) = yield strength of the transverse reinforcement
- \(\varepsilon_{su}\) = steel strain at maximum stress
- \(f'_{cc}\) = compression strength of the confined concrete

The volumetric ratio of transverse reinforcement ratio \(\rho_s\) may be approximated as:

\[
\rho_v = 1.5A_v/b_cs \quad \ldots \text{C5.10}
\]

where:
- \(A_v\) = total area of transverse reinforcement in a layer
- \(s\) = spacing of layers of transverse reinforcement
- \(b_c\) = width of column core, measured from centre to centre of the peripheral transverse reinforcement in the web

Figure C5.12 (below) shows the referred concrete stress-strain model for unconfined and confined concrete.
Note:
The original formulation of the expression for confined concrete presented by Mander et al. (1988) can predict high levels of confined concrete strain, $\varepsilon_{cu}$, depending on the assumed value for the ultimate steel strain, $\varepsilon_{su}$, of the transverse reinforcement. The modified expression suggested in fib Bulletin 25 (2003) provides a correction.

However, it is recommended that an upper bound value for the ultimate steel strain of $\varepsilon_{su} = 0.06$ (i.e. 6%) is assumed and the values of confined concrete strain are limited to $\varepsilon_{cu} = 0.015$ (1.5%) in ordinary situations.

In terms of confined concrete compression strength, $f'_{cc}$, versus unconfined concrete compression strength, $f'_{c}$, the expression presented by Mander et al. (1988) can be used:

$$ \frac{f'_{cc}}{f'_{c}} = \left( -1.254 + 2.254 \sqrt{1 + \frac{7.94f_{l}}{f'_{c}}} - 2 \frac{f_{l}}{f'_{c}} \right) $$

where $f_{l}$ is the lateral confining stress defined as:

$$ f_{l} = (TBC) $$

In lieu of more accurate analyses, when the section appears poorly confined (which is most likely to be the case for older construction) it is suggested that the confining effects on the concrete strength are neglected and $f'_{cc}/f'_{c} = 1.0$ is used instead.

In the presence of good level of transverse reinforcement and confinement action, e.g. spacing

$$ S_{max} \leq \left\{ \begin{array}{ll}
\frac{d}{2} \\
12d_{b}
\end{array} \right. $$

values of $f'_{cc} = 1.2f'_{c}$, may be assumed (Scott et al., 1982; Priestley et al., 1996).

Figure C5.13 provides charts to evaluate the confined strength ratio $f'_{cc}/f'_{c}$ as a function of the lateral confining stress.

![Figure C5.13: Enhancement of concrete compression strength due to confinement (Paulay and Priestley, 1995)](image-url)
Evaluation of moment-rotation and force-displacement curves

Once the key points of the moment-curvature of a structural element (beams, columns or walls) have been evaluated, the corresponding moment-rotation curve can be derived by integrating the curvature profile (elastic and plastic) along the cantilever scheme and after defining a plastic hinge length.

The ultimate rotation is defined as the sum of the yielding rotation and plastic rotation:

\[ \theta_u = \theta_y + \theta_p \]

where:

\[ \theta_y = \phi_y \left( \frac{H}{2} \right) \]

Yielding rotation \hspace{2cm} \text{...C5.14}

\[ \theta_p = \phi_p L_p \]

Plastic rotation \hspace{2cm} \text{...C5.15}

The force-displacement response can then be derived (Figure C5.14) by:

\[ F = \frac{M}{H} \]

\[ \Delta_u = \Delta_y + \Delta_p \]

Ultimate displacement \hspace{2cm} \text{...C5.17}

where:

\[ \Delta_y = \phi_y \frac{H^2}{3} \]

Yielding displacement \hspace{2cm} \text{...C5.18}

\[ \Delta_p = \phi_p L_p H = (\phi_u - \phi_p)L_p H \]

Plastic displacement \hspace{2cm} \text{...C5.19}

Figure C5.14: Idealisation of: (a) curvature distribution in a cantilever scheme and (b) force-displacement curve and its bilinear approximation
Plastic hinge length, $L_p$

The estimation of the plastic hinge length, $L_p$, is a delicate step in the evaluation of the ultimate rotation and displacement capacity of a member. A number of alternative formulations are available in literature to predict the plastic hinge length in beams, columns, walls and bridge piers.

The equivalent plastic hinge length, $L_p$, may be approximated (Priestley et al., 2007) as:

$$L_p = kL_c + L_{sp}$$

...C5.20

where:

$$k = 0.2 \left( \frac{f_u}{f_y} - 1 \right) \leq 0.08$$

...C5.21

$L_c$ = distance of the critical section from the point of contraflexure
$L_{sp}$ = strain penetration = $0.022f_yd_b$
$f_y$ = probable yield strength of longitudinal reinforcement
$d_b$ = diameter of longitudinal reinforcement
$f_u$ = probable ultimate strength of the longitudinal reinforcement

The first term $kL_c$ represents the spread of plasticity due to tension-shift effects and the second term $L_{sp}$ represents the strain penetration into the supporting member (e.g. beam-column joint).

**Note:**

The values presented above for the evaluation of the plastic hinge length are typically based on experimental results with reference to relatively well detailed plastic hinge regions and use of deformed bars.

However, when dealing with older construction practice, poorer detailing, low longitudinal reinforcement ratio (lightly reinforced elements), construction (cold) joints, high tensile strength of concrete, and possibly plain round bars, experimental tests as well as on-site observations from the Canterbury earthquake sequence have shown that the plastic hinge length may not develop to be as long as expected. Instead, it may be concentrated in a very short region, leading to a single crack opening and concentration of tensile strain demand in the reinforcement.

Such effects should be accounted for in the evaluation of the plastic hinge length, $L_p$, assuming much smaller values of the plastic hinge length, and assessing the effects on the overall behaviour (limited ductility/deformation capacity).

It is recommended that a plastic hinge length equal to $L_p/5$ is adopted (with $L_p$ derived from the expressions above) in the presence of either:

- plain round bars, or
- low longitudinal reinforcement ratio, i.e. $\rho_{\ell} \leq \sqrt{T_c/(4f_y)}$
- inadequately constructed cold joint, e.g. smooth and unroughened interfaces.
**Limited curvature capacity due to lap-splice failure**

The strength of lap splices in longitudinal reinforcement in plastic hinge regions will tend not to degrade during imposed cyclic loading in the post-elastic range.

In general, an available structural ductility factor of greater than 2 is not recommended if lap splices in deformed longitudinal reinforcement exist in plastic hinge regions; unless these are heavily confined.

If plain round longitudinal bars are lapped the available structure ductility factor should be taken as 1.0 (Wallace, 1996).

**C5.5.3 Columns**

**C5.5.3.1 History of code-based reinforcement requirements for columns in New Zealand**

If structural and/or construction drawings for the building are not available, it may be useful to refer to the New Zealand standards/codes of the time. Appendix C5D compares minimum design/details requirements for columns (either designed for gravity only or for seismic loading) in New Zealand according to NZS 3101:1970, 1982, 1995 and 2006. More information can be found in Niroomandi et al., 2015.

Figures C5.15 and C5.16 illustrate the evolution of structural design requirements and detailing layout for gravity column and seismic columns respectively according to the New Zealand concrete standards from the 1970s onwards.

![Figure C5.15: Example of typical gravity column layouts according to different New Zealand concrete standards from the mid-1960s on (Niroomandi et al., 2015)](image)
Part C – Detailed Seismic Assessment

C5: Concrete Buildings

C5-63

Draft Version 2016_C – 10/10/2016 NZ1-9503830-62 0.62

Figure C5.16: Example of typical column layouts with seismic design according to different New Zealand concrete standards from the mid-1960s onwards (Niroomandi et al., 2015)

The CERC report (CERC, 2012) highlighted the possibility of concrete columns not assumed to form part of the primary seismic system (referred to as gravity only columns) being inadequately detailed to accommodate the displacement demand of the building by the way in which particular clauses in the concrete structures standard NZS 3101:1982 were interpreted by designers when classifying these columns as secondary elements.

Note:

The interpretation of clause 3.5.14 of NZS 3101:1982 may have led some designers to incorrectly classify gravity columns within the general category of secondary structural elements. NZS 3101:1982 provided three options for the level of ductile detailing that was to be used in a secondary element; non-seismic provisions, seismic provisions for limited ductility, and seismic provisions.

Clause 3.5.14 specified which of these provisions should be selected, based on the level of design displacement at which the column reaches its elastic limit. If the column could be shown to remain elastic “when the design loads are derived from the imposed deformations, \( \Delta \), specified in NZS 4203”, the non-seismic provisions could be used. However, the clause was open to interpretation and in practice it appears it was applied in an inconsistent manner. Caution should be applied when making any assumptions as to the design approach that may have been employed in the original design of a building designed to these provisions.

From the mid-1980s it became more common to include the gravity system in the analysis modelling together with the seismic system. If this had been done there would be a higher chance that the secondary elements were designed with some attention to imposed deformations in mind.

In any case, it should be recognised that the imposed deformations in the design codes of the 1980s were much lower than would currently be specified. Furthermore, the deformation demand estimated from modal analysis approach (most common numerical approach used at that time) might have been inaccurate and unconservative.
Table C5.9 provides a comparison between the minimum transverse reinforcement spacing requirements of the previous standard (NZSS 1900 Chapter 9.3:1964) and the three levels of ductile detailing available in NZS 3101:1982 and subsequent versions (NZS 3101:1995 and NZS 3101:2006).

**Note:**
The primary focus of this table is on columns designed to the non-seismic and limited-ductile provisions of the 1982 standard. More detailed information on the evolution of seismic design specifications and requirements for columns in New Zealand from 1970 onwards can be found in Appendix C5D (Niromaandi et al., 2015).

<table>
<thead>
<tr>
<th>Design standard</th>
<th>Non-seismic spacing limit</th>
<th>Limited-ductile spacing limit</th>
<th>Ductile spacing limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>NZS 1900 Chapter 9.3:1964</td>
<td>For spirally-wound columns, min. of 75mm or (d_c/6)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NZS 3101:1982</td>
<td>Min. of ( h, b_c, 16d_b, 48d_{bt})</td>
<td>Min. of ( h, b_c, 10d_b, 48d_{bt})</td>
<td>Min. of ( h/5, b_c/5, 6d_b, 200 \text{ mm})</td>
</tr>
<tr>
<td>NZS 3101:1995 and NZS 3101:2006</td>
<td>Min. of ( h/3, b_c/3, 10d_b)</td>
<td>Min. of ( h/4, b_c/4, 10d_b)</td>
<td>Min. of ( h/4, b_c/4, 6d_b)</td>
</tr>
</tbody>
</table>

While the requirements for shear, anti-buckling and confinement lead to adequate transverse reinforcement detailing of the moment resisting frame (MRF) columns in NZS 3101:1982, the ‘gravity’ columns did not have matching requirements. This is a considerable oversight as the columns, while not specifically considered to contribute to the lateral force resisting mechanism, still undergo the same displacement demands as the lateral resisting system.

**Note:**
Even the 1964 standard and the non-seismic provisions in NZS 3101:1995 and 2006 required a fairly close spacing of transverse reinforcement sets. This means that columns designed using the non-seismic or limited-ductile provisions of NZS 3101:1982 are likely to be the primary concern.

It is also worth noting that the requirements in NZS 3101:1982 were more stringent for seismic conditions compared to the non-seismic and limited-ductile conditions.

There are also relevant concerns for secondary columns from other eras (pre-1982 and post-1995). This is even though the investigation by the Department of Building and Housing (now the Ministry of Business, Innovation and Employment) following the Canterbury earthquake sequence was on non-ductile columns in buildings designed to the NZS 3101:1982 (i.e. between 1982 and 1995).

In addition to low quantities of transverse reinforcement, several other characteristics of a column can contribute to its vulnerability in an earthquake. The following list provide indicative-only boundaries for key parameters that may suggest columns are susceptible to non-ductile behaviour:

- Low or inadequate quantities of transverse reinforcement – spacing (e.g. \(s > d/2\))
- High axial load demand (e.g. \(P/A_g f'_c > 0.3\))
• Low core-to-gross concrete area (e.g. \( A_c/A_g < 0.77 \))
• Detailing – inadequate lap-splice length, lap splice located in potential plastic hinge zone, poor detailing of transverse reinforcement anchorage (e.g. 90 degree bends), welded detailing, lack of support to longitudinal bars
• High inelastic inter-storey drift demand (e.g. drift > 1.5%) Location of column – in location prone to inelastic torsional amplification of displacements; e.g. corner column or column on opposite face to eccentric shear core.

This list is based on available literature and experience as proposed by (Stirrat et al., 2014). However, more experimental and numerical investigations are required to gain more confidence regarding the actual ranges.

C5.5.3.2 Probable flexural strength for columns

In general, the evaluation of the probable flexural strength for a column can follow the procedure already described for beams; with additional important considerations on the effects of axial load, due to gravity and seismic loading. These additional considerations are detailed below.

Assessors should bear in mind that collapse of all or part of a building is fundamentally due to the loss of gravity load bearing capacity of critical elements, most notably columns.

P-delta effects, bidirectional loading and other secondary effects associated to displacement compatibility issues and overall 3D response (including inelastic torsion) should be carefully considered when assessing the demand (both deformations and actions) and the capacity of columns.

Note:

This section pays particular attention to non-ductile columns, designed before NZS 3101:1995, which can potentially have inadequate detailing to sustain moderate to high levels of drift.

Development length, anchorage details and lap splices can represent potential issues in buildings designed to earlier standards. In older frames, column lap-splice connections can often be found immediately above the floor level, where the potential location of moment reversal plastic hinges cannot be precluded.

If the lap length is sufficient to develop yield (e.g. approx. \( 20d_e \) for deformed bars) then the probable flexural strength capacity can be attained. For lesser lap lengths, post-elastic deformations quickly degrade the bond strength capacity and within one inelastic cyclic of loading the lap splice may be assumed to have failed.

When the lap splice fails in bond, it does not generally lead to a catastrophic failure of the element as the column is still able to transfer moment due to the presence of the eccentric compression stress block that arises as a result of the axial load in the column. However the hierarchy of strength at that floor level can change from a weak-beam to a weak-column mechanism, potentially leading to a soft-storey.
On the other hand, premature lap-splice failure can protect from more brittle mechanism. It is thus recommended to use full flexural capacity (without reduction due to lap splice failure) when assessing shear behaviour.

The moment capacity of a lap splice \( (M_{\text{lap}}) \) may be determined as an intermediate value between the probable flexural strength assuming no deterioration, \( M_n \), capacity and a residual flexural capacity, \( (M_f) \):

\[
M_{\text{lap}} = M_n - \frac{\theta_p}{0.025} (M_n - M_f)
\]

...C5.22

where:

\( M_f \leq M_{\text{lap}} \leq M_n \)

\( \theta_p \) = plastic rotation demand on the connection

\[
M_f = \max \left( \frac{l_{d,\text{prov}}}{l_d} M_n \right)
\]

\[
0.5N(D - a)
\]

...C5.23

where:

\( l_{d,\text{prov}} \) = provided lap length

\( l_d \) = theoretical development length

\( N \) = axial load

\( D \) = the overall width of the member

\( a \) = depth of the compression stress block.

**Note:**

At a first step, and on a conservative level, the plastic rotation demand on the column, \( \theta_p \), can be taken as the one calculated for a pure flexural failure mechanism.

Similarly, the axial load force on the column can be estimated assuming a beam sway mechanism which would lead to the highest variation of the axial load.

In terms of reference values for the development length, \( l_d \), the NZS 3101:2006 recommendations for basic calculation for \( l_d \) in tension and compression can be adopted for deformed bars. For plain round bars it is recommended to take \( l_{d,\text{req}} \) as twice the specification for \( l_d \) in NZS 3101:2006.

More detailed information on bond capacity and development length of plain round bars can be found in (Fabbrocino et al., 2002).

In general, the refined calculations for \( l_d \) in NZS 3101:2006 are not applicable to older construction practice; in particular, to pre-1970s columns with inadequate confinement transverse reinforcement. The following adjustments are suggested.

The basic development length in tension is given by:

\[
l_d = \frac{0.5\psi_a f_y}{f'_c} d_b
\]

...C5.24

where \( \psi_a = 1.3 \) for beam top reinforcement with at least 300 mm concrete underneath the bars and 1.0 for all other cases. \( f'_c \) in Equation C5.24 is limited to 70 MPa.
The basic development length in compression is given by:

\[ l_d = \frac{0.22 f_y d_b}{\sqrt{f'c}} > 0.04 f_y d_b > 200 \text{mm} \]  ...C5.25

In terms of the development length of bars with hook anchorage (as in anchorage of beam longitudinal reinforcements into the beam-column joint), NZS 3101:2006 provides a different \( l_d \) equation:

\[ l_d = 0.24 \Psi_b \Psi_1 \Psi_2 \frac{f_y d_b}{\sqrt{f'c}} > 8 d_b \]  ...C5.26

where:

\[ \Psi_b = \frac{A_{s,req}}{A_{s,prov}} \text{ in the column,} \]
\[ \Psi_1 = 0.7 \text{ for } 32\text{mm} \; d_b \text{ or smaller with side concrete cover } \geq 60 \text{ mm and hook end cover } \geq 40 \text{ mm, and} \]
\[ \Psi_2 = 1.0 \text{ for other cases, and} \]
\[ \Psi_2 = 0.8 \text{ for well confined splice (with stirrups spacing } < 6 d_b) \text{ and} \]
\[ \Psi_2 = 1.0 \text{ for other cases.} \]

\( A_{s,req}, A_{s,prov} \) are the area of flexural reinforcements required and provided, respectively.

### C5.5.3.3 Probable shear strength capacity of columns

The probable shear strength capacity of columns outside the plastic hinge region can be taken as:

\[ V_p = 0.85(V_c + V_s + V_n) \]  ...C5.27

where \( V_c, V_s \) and \( V_n \) are the shear contributions provided by the concrete mechanism, steel shear reinforcement and the axial compressive load \( N \) (shown in some figures as \( P \)) respectively.

In more detail:

- the shear contribution from the concrete, \( V_c \), can be evaluated as:

\[ V_c = \alpha \beta \gamma \sqrt{f'c} \left(0.8 A_g \right) \]  ...C5.28

where:

\[ 1 \leq \alpha = 3 - \frac{M}{V D} \leq 1.5 \]
\[ \beta = 0.5 + 20 \rho_l \leq 1 \]
\[ \gamma = \text{shear strength degradation factor (refer to Figure C5.17)} \]
\[ A_g = \text{gross area of the column} \]
\[ M/V = \text{ratio of moment to shear at the section} \]
\[ D = \text{total section depth or the column diameter as appropriate} \]
\[ \rho_l = \text{area of longitudinal column reinforcement divided by the column cross-sectional area}. \]
- The shear contribution from the steel shear reinforcement, $V_s$, is evaluated assuming that the critical diagonal tension crack is inclined at $30^\circ$ to the longitudinal axis of the column.

For rectangular hoops:

$$V_s = \frac{A_v f_{yt} d^\prime}{s} \cot 30^\circ \quad \ldots \text{C5.29}$$

and for spirals or circular hoops:

$$V_s = \frac{\pi A_{sp} f_{yt} d^\prime}{2s} \cot 30^\circ \quad \ldots \text{C5.30}$$

where:

- $A_v$ = total effective area of hoops and cross ties in the direction of the shear force at spacing $s$
- $A_{sp}$ = area of spiral or circular hoop bar
- $f_{yt}$ = expected yield strength of the transverse reinforcement
- $d^\prime$ = depth of the concrete core of the column measured in the direction of the shear force for rectangular hoops and the diameter of the concrete core for spirals or circular hoops.

- The shear resisted as a result of the axial compressive load $N^*$ on the column is given by:

$$V_n = N^* \tan \alpha \quad \ldots \text{C5.31}$$

where $\alpha$ is:

- for a cantilever column, the angle between the longitudinal axis of the column and the straight line between the centroid of the column section at the top and the centroid of the concrete compression force of the column section at the base (refer to Figure C5.17a)
- for a column in double curvature, the angle between the longitudinal axis of the column and the straight line between the centroids of the concrete compressive forces of the column section at the top and bottom of the column (refer to Figure C5.17b).

![Diagram of shear strength assessment](image)

**Figure C5.17:** Column shear strength assessment based on Priestley et al. (1994, 1995, 2007)
Note:
As mentioned earlier in the case of beams, the degradation of the shear strength in a plastic hinge regions is affected by the ductility demand and cyclic loading.

Figure C5.17 shows proposals for the degradation of the nominal shear stress carried by the concrete, \( \gamma \) [when using MPa Units], as a function of the imposed ductility factor \( \phi/\phi_y \), as proposed for columns by Priestley et al. (1994, 2007).

**C5.5.3.4 Internal hierarchy of strength and sequence of mechanisms in a column**

Once the various failure mechanisms for a column have been evaluated, including flexural, shear, lap-splice failure and bar buckling, the (force-based) hierarchy of strength and expected sequence of events can be visualised within an M-N interaction diagram or performance-domain (Pampanin et al., 2002) in order to account for the variation of axial load during the frame sway mechanism.

As an example of the M-N interaction diagram for a column with poor detailing Figure C5.18 shows:
- conventional tensile and compressive flexural failures
- shear capacity/failure and shear degradation at various ductility levels (\( \mu = 2 \) and \( \mu = 4 \))
- lap-splice failure of the column longitudinal reinforcement.

![Figure C5.18: Internal hierarchy of strength of column failure modes within an M-N interaction diagram (Kam, 2011)](image)

Such force-based hierarchy of strength and sequence of event information should be integrated with the information on the rotation or displacement capacities associated with each mechanism, as discussed in the following Section C5.6.

Ultimately, by combining the flexural capacity curve with the shear degradation capacity curve, an overall force-displacement capacity curve for the column can be derived and will highlight the occurrence of the various mechanisms at different curvature/rotation/displacement (and therefore the interstorey drift) level, as shown in Figure C5.19.
C5.5.3.5 Deformation (curvature, rotation and displacement/drift) capacities of columns

Plastic rotation and displacement/drift capacity due to flexural mechanism

The procedure outlined in Section C5.5.2.5 to evaluate the rotation capacity of a plastic hinge for beams also applies, with minor changes, to plastic hinges forming at column bases, or in column sidesway mechanisms. However, the approximation for the volumetric ratio of transverse reinforcement, $\rho_s$, in Equation C5.10 should be replaced by a first principles approach. In fact, it will often be found that columns in older reinforced concrete frames have only nominal transverse reinforcement so must be considered to be unconfined.

Together with reduced plastic hinge length as a consequence of reduced member height compared with beam length, and reduced ultimate curvature as a consequence of axial compression, column plastic rotation capacity will generally be less than the values estimated for beams. Values less than $\theta_p = 0.01$ radians will be common for unconfined situations and $\theta_p = 0.015$ for confined situations. The ultimate (ULS) rotation capacity will be given by the sum of the elastic and plastic contributions.

It is worth remembering that the axial load level critically affects the ultimate curvature and thus the ultimate rotation capacity of a column. A proper estimation of the expected level of axial load due to gravity loads and the variation due to the application of lateral seismic loads should be carried out. More details are provided in the following sections on beam-column joints, hierarchy of strength and determination of the ‘seismic’ axial load contribution from a frame sway mechanism.

In fact, from a rotation capacity point of view the critical column will be the one with highest axial compression, while from a moment capacity point of view the critical column will be the one with the lowest axial load.
Note:

Moment-curvature analyses will show that, while the yield curvature is not greatly affected by axial load level (particularly when yield curvature is expressed in terms of equivalent elasto-plastic response), the ultimate curvature and hence plastic rotation capacity is strongly dependent on axial load.

This is illustrated in Figure C5.20, where a poorly confined (transverse reinforcement D10@400, 2 legs only) end column of a frame with nominal axial load of \( P = 0.2f'_c A_g \) is subjected to seismic axial force variations of \( P_E = \pm 0.2f'_c A_g \). The yield curvatures differ by less than 10% from the mean, while the ultimate curvatures at \( P = 0 \) and \( P = 0.4f'_c A_g \) are 61% and 263% of the value at \( P = 0.2f'_c A_g \).

![Figure C5.20: Moment-curvature response of a column with poor confinement](image)

**Displacement/drift capacity due to flexure-shear failure mechanism**

Exceeding the shear capacity of RC columns in a flexure-shear mode does not necessarily imply loss of axial load carrying capacity. In such a mixed mode, when shear capacity is exceeded, axial load can still be supported by the longitudinal reinforcing bars and force transfer through shear friction.

When a column behaviour is characterised by a flexural-shear behaviour with shear strength reduction due to ductility demand, the ultimate displacement capacity can be estimated as the point at which the probable shear strength degradation curve intersects the probable flexural strength curve.

The displacement of a column at the point that the shear capacity is reached, \( \Delta_s \), can be roughly estimated from (Elwood and Moehle, 2005). In the context of these guidelines, \( \Delta_s \), is to be considered as the probable drift/displacement based limit associated with the evaluation of %NBS:

\[
\Delta_s = L_c \left( 0.03 + 4\rho_s - 0.024 \frac{v}{\sqrt{f'_c}} - 0.025 \frac{\rho}{A_g f'_c} \right) \geq 0.01L_c \quad \ldots C5.32
\]
Details-dependent drift levels are calculated for the yielding of the section, shear failure and post shear-failure loss of axial load carrying capacity.

**Note:**
Shear mechanisms, particularly the post-peak displacement behaviour of columns dominated by shear failure mechanisms, is a complex area of research that is still under development. Different models have been proposed (e.g. Elwood and Moehle, 2005; Yoshimura, 2008), which can provide a significant scatter in terms of predicted values.

Given the dramatic impact that shear failure of columns in particular can have, as this can lead to loss of gravity bearing capacity, it is recommended that the assessment of their ultimate capacity is treated with care and that specific remedial (retrofit) interventions are considered to eliminate such potentially severe critical structural weaknesses (CSWs).

### Displacement/drift capacity due to bar buckling

The ultimate capacity of a flexure-governed non-ductile column could be limited by longitudinal bar buckling before the initiation of concrete crushing.

The expression proposed by Berry & Eberhard (2005) can be employed to estimate the lateral displacement $\Delta u$ at which buckling of the longitudinal bars of this type of column is initiated.

$$\Delta u = 0.0325 L_c \left(1 + k_{e_{bb}} \rho_{eff} \frac{d_b}{D} \right) \left(1 - \frac{p}{A_{g} f'_{c}} \right) \left(1 + \frac{L_{c}/2}{10d} \right)$$  \(\text{C5.33}\)

where:

- $k_{e_{bb}} = 0$ for columns with $sd_b \geq 6d_b$
- $k_{e_{bb}} = 40$ and $150$ for rectangular columns and spiral-reinforced columns, respectively.

**Note:**
It is worth noting that the original expression proposed by Berry & Eberhard (2005) was calibrated on the drift ratio ($\Delta u/L_c$) obtained from experimental results. The dispersion of such expressions, when applied directly to derive displacement, is quite high and should be treated with care.

### C5.5.3.6 Non-ductile “gravity” columns

The capacity of non-ductile “gravity” columns, which are described in Section C5.3, should be assessed in the same manner as that recommended above for columns forming part of the lateral load resisting system.

However, given their critical role of gravity-load-carry capacity and the lack of adequate detailing which could lead to brittle failure mechanisms, special care should be taken when assessing their capacity and performance. This acknowledges the higher level of uncertainty in the prediction of displacement/drift capacity associated with shear failure, particularly when bidirectional loading is considered.
C5.5.4 Beam-column joints

C5.5.4.1 History of code-based reinforcement requirements for beam-column joints

If structural and/or construction drawings for the building are not available it may be useful to refer to the requirements of the New Zealand standards of the time. Appendix C5D summarises the minimum design requirements for beam-column joint reinforcement and details according to NZS 3101:1970, 1982, 1995 and 2006. More information can be found in Cuevas et al. (2015).

Figure C5.21 illustrates the evolution of structural design requirements and detailing layout for beams according to these standards.

Figure C5.21: Example of typical beam-column joint layouts according to different New Zealand standards (Cuevas et al., 2015)

C5.5.4.2 Typical deficiencies in beam-column joint design and detailing

Older RC buildings can be characterised by a number of different construction practices and structural detailing for beam-column connections. Typical inadequacies can be related to the:

- lack or absence of horizontal and/or vertical transverse reinforcement
- non-ductile anchorage of beam longitudinal bars into the joint, and
- lack of reliable joint shear transfer mechanism beyond diagonal cracking.

The primary deficiency of older beam-column joints, particularly before the 1970s, was the inadequate joint shear reinforcement. In fact, in older construction practice beam-column joints were treated either as construction joints or as part of the columns. Consequently, these beam-column joints would have no, or very few, joint stirrups.

As demonstrated in laboratory testing (Hakuto et al., 2000; Pampanin et al., 2002-2003) and post-earthquake observations, different types of damage or failure modes are expected to occur in beam-column joints depending on the:

- typology (i.e. exterior or interior joints, with or without transverse beams) and
• structural details; i.e.:
  - lack or insufficient transverse reinforcement in the joint
  - type of reinforcement, i.e. plain round or deformed
  - alternative bar anchorage solutions; i.e. bent in, bent out, end-hooked, or a combination of these.

Figure C5.22 illustrates possible damage mechanisms of exterior tee-joints with no or minimal transverse reinforcement in the joint regions and alternative beam anchorage details.

Alternative damage mechanisms for exterior tee-joints are shown in Figure C5.22:
• beam bars bent inside the joint region – (a) and (b)
• beam bars bent outside the joint region – (c), and
• plain round beam bars with end-hooks: “concrete wedge” mechanism – (d).

All of these solutions have been used in New Zealand.

![Figure C5.22](image)

Figure C5.22: Alternative damage mechanisms expected in exterior joints depending on the structural detailing: (a) and (b) beam bars bent inside the joint region; (c) beam bars bent outside the joint region; (d) plain round beam bars with end-hooks

Note:
Referring to the basic strut-and-tie theory for beam-column joints (Park and Paulay, 1975; Paulay and Priestley, 1995) shown earlier in Figure C5.6, it is expected that exterior joints of older construction practice (i.e. with poor or no transverse reinforcement in the joints and poor anchorage detailing of the beam bars) are usually more vulnerable than interior beam-column joints.

After diagonal cracking, the shear transfer mechanism in a joint with no or very limited shear reinforcement must essentially rely on a compression diagonal strut. This mechanism can be maintained up to a certain level of compression stress in an interior beam-column joint. However, when dealing with exterior beam-column joints the strut efficiency is critically related to the anchorage solution adopted for the longitudinal beam reinforcement.

When the beam bars are bent into the joint (refer to Figure C5.22(a) and (b)) they can provide a limited resistance against the horizontal expansion of the joint. This is until the hook opens
under the combined action of the diagonal strut and the pulling tension force in the beam reinforcement, which then leads to a rapid joint degradation. When the beam bars are bent away from the joint (refer to Figure C5.22(c)), as is more typical of older construction practice in New Zealand, no effective node point is provided for the development of an efficient compression strut mechanism unless a significant amount of transverse column hoops is placed immediately above the joint core. In this case, rapid joint strength degradation after joint diagonal cracking is expected.

Arguably, the worst scenario is provided by the solution shown in Figure C5.22(d), which is more common in pre-1970s buildings and consists of plain round bars with end-hook anchorage. The combination of an inefficient diagonal strut action and a concentrated compression force (punching action) at the end-hook anchorage due to slippage of the longitudinal beam bars can lead to the expulsion of a ‘concrete wedge’ and rapid loss of vertical load capacity.

### C5.5.4.3 Probable shear strength of beam-column joints

#### Joints without any shear reinforcement

For interior and exterior beam-column joints without shear reinforcement, as typical of pre-1970s buildings, the probable horizontal joint shear force that can be resisted is:

\[
V_{pjh} = 0.85v_{ch}b_jh
\]

\[
= 0.85k\sqrt{f'_c\left(1 + \frac{N^*}{A_gk'f'_c}\right)b_jh} \leq 1.92f'_cb_jh
\]

where:

- \(v_{ch}\) = nominal horizontal joint shear stress carried by a diagonal compressive strut mechanism crossing the joint
- \(b_j\) = effective width of the joint (being normally the column width as per NZS 3101:2006)
- \(h\) = depth of column.

The following values for \(k\) should be used:

- for interior joints, \(k = 0.8\) (note that compression failure rather than tensile failure would govern in an interior beam-column joint)
- for exterior joints with beam longitudinal (deformed) bars anchored by bending the hooks into the joint core, \(k = 0.4\)
- for exterior joints with beam longitudinal (deformed) bars anchored by bending the hooks away from the joint core (into the columns above and below), \(k = 0.3\)
- for exterior joints with beam longitudinal (plain round) bars anchored with end hooks, \(k = 0.2\).

**Note:**

These recommended values for \(k\) are based on experimental testing from Hakuto et al., 1995-2000 (mostly focusing on deformed bars with no variation of axial load) and Pampanin et al., 2000-2010 (mostly focusing on plain round bars and variation of axial load).
\(v_{ch}\) indicates the estimated maximum nominal horizontal joint core shear stress, calculated the conventional way, resisted by beam-column joints in tests without joint shear reinforcement and without axial load.

The term indicating the influence of axial load, \(\sqrt{1 + \frac{N}{A_g f_c'}}\) was obtained by assuming that the diagonal (principal) tensile strength, \(p_t\), of the concrete was \(p_t = k_f' f'_c\) and using Mohr’s circle to calculate the horizontal shear stress required to induce this diagonal (principal) tensile stress when the vertical compressive stress is \(N^*/A_g\) [Hakuto et al (2000), Pampanin et al., (2002)].

A strength reduction factor of 0.85 has been included in Equation C5.34 to account for the higher uncertainty (and impact) of a shear failure mechanism when compared to a flexural one.

In fact, it has been demonstrated (Priestley, 1997; Pampanin, 2002) that principal tensile and compression stresses, \(p_t\) and \(p_c\), are more appropriate indicators of joint damage than the nominal shear stress \(v_{jh}\), as they can take the variation of axial load into account.

Principal tensile stresses, \(p_t\), would tend to govern the failure mechanism of exterior beam-column joints (tensile cracking), while principal compression stresses, \(p_c\), would tend to govern interior beam-column joints where higher levels of axial load are expected and the damage/failure mechanism is more correlated to the degradation of the diagonal compression strut.

Figure C5.23 shows strength degradation curves \(p_t\) versus \(\gamma\) (shear deformation) as well as \(p_t\) versus drift presented in literature and based on extensive experimental tests.

![Figure C5.23: Strength degradation curves for exterior joints (Pampanin et al., 2002)](image_url)

Indicative level of damage limit states for exterior beam column joints with no shear reinforcement, expressed in terms of shear deformation, \(\gamma\) [rad], and interstorey drift, \(\theta\), are reported in Table C5.10. For the scope of this document and assessment procedure, the critical damage limit state can be considered as the ultimate limit state for the joints, to be used for the evaluation of the %NBS of the building.
In the case of interior joints, given the possibility to develop a joint shear transfer mechanism via diagonal compression strut the limit states values of Table C5.10 can be increased by approximately 50%.

**Table C5.10: Suggested limit states for exterior joints with no shear reinforcement (modified after Magenes and Pampanin, 2004)**

<table>
<thead>
<tr>
<th>Limit states</th>
<th>Extensive damage</th>
<th>Critical damage (corresponding to ULS)</th>
<th>Incipient collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear deformation, ( \gamma ) [rad]</td>
<td>( 0.005 \leq \gamma &lt; 0.01 )</td>
<td>( 0.01 \leq \gamma &lt; 0.015 )</td>
<td>( \gamma \geq 0.015 )</td>
</tr>
<tr>
<td>Drift, ( \theta ) [%]</td>
<td>( 0.8% \leq \theta &lt; 1.2% )</td>
<td>( 1.2% \leq \theta &lt; 1.8% )</td>
<td>( \theta \geq 0.02 )</td>
</tr>
</tbody>
</table>

**Note:**
The limit states proposed above are based on experimental and numerical investigations on beam-column joint subassemblies and frame systems.

It is worth noting that the interstorey drift corresponding to a specific damage level in the joint panel zone would depend on the elastic and plastic contribution of beams and column and thus would need to be checked on a case-by-case basis.

**Joints with some shear reinforcement**

For interior and exterior beam-column joints with some shear reinforcement (stirrups), the probable horizontal joint shear force that can be resisted is:

\[ V_{pjh} = 0.85v_{jh}b_jh \] …C5.35

For joints with interior stirrups the joint shear stress can be computed, based on similar considerations on Mohr’s Circle approach, as:

\[ v_{jh} = 0.85 k \sqrt{f'c} \sqrt{1 + k \sqrt{f'c} (f_v + f_h) + f_v f_h} \] for exterior joints …C5.36

\[ v_{jh} = 0.85 k f'c \sqrt{1 + k f'c (f_v + f_h) + f_v f_h} \] for interior joints …C5.37

where:

\[ f_v = \frac{N}{A_g} \] is the axial load stress on the joint

\[ f_h = \frac{A_{stf_{sy}}}{b_jh} \] represents horizontal confinement effects due to the stirrups in the joint and is calculated as the maximum tension stress that the stirrups can develop at yield.

**Note:**
The expression above is used in Eurocode 8 to determine the required amount of stirrups in a joint.
For \( f_h = 0 \) (and after substituting the definition of principal tensile stress, \( p_t \), as a function of nominal shear stress, \( \nu_{jh} \) and the axial load stress \( f_v \)) the general equation for joints with shear reinforcement converge to the equation for joints with no shear reinforcement.

Taking a rigorous approach, the joint capacity would be evaluated considering both principal tensile and compression stresses approach. However, in practical terms and considering that exterior joints are mostly governed by tensile cracking failure and interior joints by compression (crushing) failure, the expression presented above (based on principal tensile stress \( p_t = k\sqrt{f'_c} \)) can be used for exterior joints.

For interior joints a similar expression based on principal compression stresses is obtained by replacing \( p_t = k\sqrt{f'_c} \) with \( p_c = k_c f'_c \) and assuming \( k = 0.6 \) for critical damage level.

### C5.5.4.4 Procedure for evaluating the equivalent “moment” capacity of a joint, \( M_j \)

In order to compare the hierarchy of strength and determine the expected sequence of events within beam–column joint subassemblies (refer to Section C5.6.1 for full development) the joint shear capacity needs to be expressed as a function of a comparable parameter to the capacity of beams and columns.

As a benchmark parameter, it is suggested to take the equivalent moment in the column (based on equilibrium considerations) corresponding to the selected limit states, e.g. first cracking, extensive damage/peak capacity in the joint.

In Table C5.11 and Figure C5.24 below the nominal shear force \( V_{jh} \) is expressed as a function of the moment in the column, leading to the expression of \( M_{col} \) as the equivalent moment in the column corresponding to a given event or damage in the joint panel zone.
Table C5.11: Step-by-step procedure to express the joint capacity as a function of equivalent column moment $M_j$ or $M_{col}$

<table>
<thead>
<tr>
<th>Procedure</th>
<th>Equation</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Horizontal shear force acting on the joint core</strong></td>
<td>$V_{jh} = T - V_c$</td>
<td>C5.38</td>
</tr>
<tr>
<td><strong>Equilibrium of the external action</strong></td>
<td>$V_{jc} = V_b l_b$</td>
<td>C5.39</td>
</tr>
<tr>
<td><strong>Rearrange to get $V_b$</strong></td>
<td>$V_b = \frac{V_{jc}}{l_b}$</td>
<td>C5.40</td>
</tr>
<tr>
<td><strong>Moment acting at the face of the joint core</strong></td>
<td>$M_b = V_b \left( l_b - \frac{h_c}{2} \right) = T_j d$</td>
<td>C5.41</td>
</tr>
<tr>
<td><strong>Rearrange to get $T$</strong></td>
<td>$T = \frac{M_b}{j d} = \frac{V_b (l_b - \frac{h_c}{2})}{j d} = \frac{V_{jc} (l_b - \frac{h_c}{2})}{l_b j d}$</td>
<td>C5.42</td>
</tr>
<tr>
<td><strong>Substitute into the 1st equation</strong></td>
<td>$V_{jh} = T - V_c = \frac{V_{jc} (l_b - \frac{h_c}{2})}{l_b j d} - V_c \left[ \frac{l_c}{l_b j d} \left( l_b - \frac{h_c}{2} \right) - 1 \right]$</td>
<td>C5.43</td>
</tr>
<tr>
<td><strong>Rearrange to get $V_c$</strong></td>
<td>$V_c = \left[ \frac{l_c}{l_b j d} \left( l_b - \frac{h_c}{2} \right) - 1 \right]$</td>
<td>C5.44</td>
</tr>
<tr>
<td><strong>Joint capacity in terms of the column moment</strong></td>
<td>$M_{col} = V_c \left( l_c - \frac{h_b}{2} \right) = \frac{V_{jh}}{l_c j d} \left( l_c - \frac{h_b}{2} \right)$</td>
<td>C5.45</td>
</tr>
<tr>
<td><strong>Assume $j = 0.9 d$ and $A_e = b_j \times h_c$</strong></td>
<td>$M_{col} = v_{jh} (1000) kNm$ and $\phi = \frac{2l_c^2 - 1.8 d l_b}{0.9 d l_b A_e (l_c - h_b)}$</td>
<td>C5.46</td>
</tr>
<tr>
<td><strong>Nominal horizontal shear stress at the mid-depth of the joint core</strong></td>
<td>$v_{jh} = \frac{V_{jh}}{b_j \times h_c}$</td>
<td>C5.47</td>
</tr>
<tr>
<td><strong>Effective width of the joint</strong></td>
<td>$b_j = \min (b_c, b_w + 0.5 h_c)$ if $b_c \geq b_w$</td>
<td>C5.48</td>
</tr>
<tr>
<td></td>
<td>$b_j = \min (b_w, b_c + 0.5 h_c)$ if $b_c \leq b_w$</td>
<td>C5.49</td>
</tr>
<tr>
<td><strong>Principal tensile and compressive stresses</strong></td>
<td>$p_t = p_c = -\frac{f_v}{2} \pm R$</td>
<td>C5.50</td>
</tr>
<tr>
<td><strong>Substitute $R = \sqrt{\left( \frac{f_v}{2} \right)^2 + v_{jh}^2}$ from Mohr’s Circle Theory</strong></td>
<td>$p_t = -\frac{f_v}{2} + \sqrt{\left( \frac{f_v}{2} \right)^2 + v_{jh}^2}$</td>
<td>C5.51</td>
</tr>
<tr>
<td><strong>Rearrange to get horizontal shear</strong></td>
<td>$v_{jh} = \sqrt{p_t^2 + p_t f_v}$</td>
<td>C5.52</td>
</tr>
<tr>
<td><strong>Substitute into the joint capacity equation</strong></td>
<td>$M_{col} = \sqrt{p_t^2 + p_t f_v (1000)} kNm$</td>
<td>C5.53</td>
</tr>
<tr>
<td><strong>Principal tensile stress</strong></td>
<td>$p_t = k \sqrt{f_c}$</td>
<td>C5.54</td>
</tr>
<tr>
<td><strong>Stress due to axial load</strong></td>
<td>$f_v = \frac{N_v}{A_e}$</td>
<td>C5.55</td>
</tr>
</tbody>
</table>
Figure C5.24: (a) Free-body diagram of a beam-column joint subassembly; (b) Mohr’s circle theory applied to calculate joint shear and principal tensile/compression stresses; (c) Moment, shear and stresses at joint region (modified after Pampanin et al., 2003; Akguzel and Pampanin, 2010; Tasligedik et al., 2015)
For an interior joint the same procedure can be followed by:

- introducing the contribution from the compression steel, $C's$, of the other beam in the first equation in Table C5.11:

$$ V_{jh} = T + C's - V_c \quad \ldots C5.56 $$

- assuming $M_b = M_c$ for interior beam-column joints, instead of $M_b = 2M_c$ for exterior joints, and
- checking that $l_b'$ and $l_b$ are to be taken as the beam clear span and full span respectively, consistent with an interior beam-column joint.

**Note:**

This procedure is intended to be a simple analytical approach to determine the hierarchy of strength and the global mechanism as part of a SLaMA method. The full procedure to evaluate the hierarchy of strength and sequence of events for a beam-column joint subassembly is presented in Section C5.6.1.

The example provided assumes a point of contraflexure at mid-height of the column, which might in fact vary during the sway mechanism; in particular when yielding columns or joint shear damage and failure occur at one level requiring redistribution and due to the dynamic effects.

Refer to Section C2 for more information on the limitations of alternative analysis methods.

**C5.5.4.5 Effects of bidirectional cyclic loading on joint capacity**

The effects of bidirectional loading can significantly affect the response of poorly detailed beam-column joints and modify the hierarchy of strength and sequence of events of the subassembly – and thus possibly the overall global response of the frame.

Conceptually, the shear (or equivalent moment) strength reduction due to bidirectional loading is similar to that expected in a column (both in flexure and shear) when subjected to bidirectional loading (refer to Figure C5.25).

![Figure C5.25: Conceptual moment-axial load ($M_y - M_z - P$) or shear-axial load ($V_y - V_z - P$) interaction surface for a reinforced concrete element (including beam-column joint) subjected to bi-axial loading](image-url)
In the absence of more detailed study or evidence, a reduction of 30% on the joint shear (strength) capacity within the subassembly hierarchy of strength should be considered under bidirectional loading. Also, it is suggested that the lower bounds of the deformation limit states indicated in Table C5.10 are adopted to account for the effect of bidirectional loading.

Note:
Most of the available studies available on the seismic assessment and retrofit of existing poorly detailed frame buildings have concentrated on the two-dimensional response, thus subjecting the specimen or subassemblies to unidirectional cyclic loading testing protocols. Even when the 3D response under combined bidirectional loading has been taken into account in experimental testing, the focus has been typically on interior (fully or partially confined) joints.

As part of a more extensive research programme on seismic retrofit solutions for New Zealand RC buildings, the effects of bidirectional loading – which is more representative of the actual seismic response of a building structure – on the assessment and design of the retrofit intervention have been investigated (Akguzel and Pampanin, 2010).

These results confirmed that the bidirectional cyclic loading can significantly affect the response of poorly detailed beam-column joints (with a reduction of the lateral load capacity of the whole subassembly of approximately 30%).

Figures C5.26 and C5.27 show an example of the observed damage and a comparison of the subassemblies’ hysteresis loops.

In both 2D and 3D specimens a shear hinge mechanism developed in the joint region, providing the main source of the observed inelastic deformation and behaviour.

However, the 3D specimen exhibited a more complex three-dimensional concrete wedge mechanism (Figure C5.26(b)), well in line with the damage observed in recent earthquake events. A critical level of joint damage and a more rapid strength degradation were observed when compared to the 2D equivalent (Figure C5.26(a)), in spite of the partial confinement effects provided by the orthogonal beam.

In presence of bidirectional loading it is thus recommended to account for a reduction of both strength and deformation capacities in the joints.
Figure C5.26: Damage observation from laboratory testing in the as-built exterior beam-column joint specimens 2D and 3D subjected to uni- and bidirectional loading respectively (after Pampanin, 2009; Akguzel and Pampanin, 2010)

Figure C5.27: Experimental hysteresis and envelope curves for two exterior 2D and 3D exterior joint subassemblies subjected to unidirectional and bidirectional loading regime respectively (after Akguzel and Pampanin, 2010)

Note:
Overlooking the effects of bidirectional loading on the local and global response and the performance of an RC structure can significantly impair the efficiency of a retrofit intervention.

As for the variation of axial load, a controversial outcome could be that an (inappropriately) selected retrofit intervention would actually lead to a global failure mechanism (i.e. due to the formation of a soft storey) which may not have occurred in the as-built (pre-retrofit) configuration.
C5.5.5 Structural walls

C5.5.5.1 History of code-based reinforcement requirements for walls

If structural and/or construction drawings for the building are not available it may be useful to refer to requirements of the New Zealand standards of the time. Refer to Appendix C5D.5 for a comparison of minimum design and detail requirements for walls according to NZS 1900:1964 and NZS 3101:1970, 1982, 1995, 2006. More information can be found in Dashti et al., 2015.

Figure C5.28 illustrates an example of the evolution of structural design requirements and detailing layout for shear walls according to these standards.
Figure C5.28: Example of typical reinforcement layouts for shear walls designed according to different New Zealand concrete standards from mid-1960s on (Dashti et al., 2015)
C5.5.5.2 Failure mechanisms for shear walls

Depending on the geometric and mechanical characteristics (reinforcing details and layout) and on the demand (unidirectional or bidirectional, level of axial load and moment/shear), structural (shear) walls can develop alternative and complex mechanisms as demonstrated in extensive experimental testing in structural laboratories as well as by damage observed following major earthquakes.

Figure C5.29 gives an overview of the most commonly expected and analysed failure mechanisms in shear walls under unidirectional loading (Paulay, 1981).

In addition to the most desirable flexural yielding of the longitudinal reinforcement in the plastic hinge region (b), alternative failure modes such as diagonal tension (c) or diagonal compression due to shear, instability of thin walled sections or buckling of the main compression reinforcement (refer to Appendix C5G), sliding-shear along the construction joints (d) and shear or bond failure along lapped splices or anchorage can occur and should be assessed.

Poor or inadequate detailing can lead to a severe and sudden strength degradation; potentially at relatively low levels of lateral displacement/drift demand.

![Figure C5.29: Various failure modes of cantilevered shear walls (Paulay, 1981)](image)

Note:

Concrete walls in buildings constructed before the importance of the ductile capacity was recognised will typically have low levels of shear and confinement reinforcing.

Anti-buckling and confinement stirrups and ties were not required before NZS 3101:1982. Compression zone ductile detailing was introduced at that time, with specific requirements to limit the extreme fibre compressive strain or provide boundary confining stirrups.

Furthermore, pre-1970s concrete walls were typically constructed as infill panels in between concrete columns and perforated with multiple openings. Typical pre-1970s walls (for low to mid-rise buildings) were 6” to 8” thick (approx. 150-200 mm) and lightly reinforced with 3/8” or ¼” bars at 8” to 12” centres (approx. 200-300 mm). However, the increase in flexural capacity of the wall including the longitudinal reinforcement of the boundary columns may result in increased shear demands and a brittle shear-dominated inelastic mechanism.
The major Chile earthquake of 2010 and the Canterbury earthquake sequence of 2010-2011 provided real examples of most, if not all, of the ‘traditional’ mechanisms referred to earlier (NZSEE 2010-2011 and EERI/NZSEE 2015 Special Issues).

In addition, a number of alternative failure mechanisms have been highlighted as shown in previous sections. These include:

- out-of-plane instability of doubly reinforced, well confined and not necessarily “thin” (as typically considered) walls
- diagonal compression-shear failure of walls due to interaction (displacement compatibility) with the floor system during the uplifting
- out-of-plane shear/sliding failure at lap-splice level, in part due to bidirectional loading effects, and
- flexural tension failure of singly reinforced walls with low-reinforcement ratio.

The key parameters controlling the behaviour and alternative mechanisms of walls are both geometrical and mechanical:

- element shear span ratio \( (V/M) \), i.e. squat vs. tall
- section aspect ratio \( (L_w/t_w) \)
- slenderness ratio \( (H/t_w) \)
- longitudinal reinforcement ratio in the boundary elements and in the core \( (\rho_l) \)
- transverse reinforcement and confinement details in the boundary regions, and
- axial load ratio \( (N/f'_cA_c) \).

**Note:**

Following observations of the relatively poor performance of existing walls in the aftermath of the Chile and Canterbury earthquakes, there is an ongoing and internationally coordinated research effort under the name of “Wall International Institute”. The purpose of this research, which is based on experimental, numerical and analytical investigations, is to improve the understanding of shear wall building behaviour (at local, member and global system level) in order to refine current provisions both for new design and the assessment of existing walls (Wallace et al., 2016).

The methods described in these guidelines (either in the core text and in the appendices) are based on the latest knowledge and will be updated as new research evidence becomes available in the near future.

**C5.5.5.3 Probable flexural strength of walls**

**General approach**

In general terms, the evaluation of the probable flexural strength for a shear wall at the critical sections can follow the procedure described earlier for columns, with some additional considerations for alternative failure mechanisms. These are highlighted below.

As outlined for columns, the actual axial load demand due to gravity loads and seismic action should be accounted for.
The probable flexural strength, $M_{wp}$, of each wall should be determined based on the effective vertical reinforcement at the base and the gravity loads. The neutral axis depth to wall length ratio, $c/l_w$, which is derived as a by-product of this calculation, is used subsequently when checking the curvature ductility capacity of each wall section.

**Note:**
As a first approximation, a traditional section analysis can be carried out. This should take into account the distributed reinforcement and assume a linear strain profile based on “plane sections remaining plane” assumption and a full bond condition between the steel rebars and the concrete.

However, it has recently been shown that depending on the structural detailing and key mechanical/geometrical parameters such an assumption of linear strain profile might not be valid; particularly for post yield behaviour. Strain and stress concentrations (both tension and compression) can thus occur and develop not only along the section depth but also across the thickness, leading to more complex out-of plane or localised failure mechanisms as outlined in Appendix C5G. More information can be found in Dashti et al., 2015.

**Lap-splice connection failure and bond slip**

In older shear walls, lap splice often occurs within the plastic hinge regions and can develop for a significant length (e.g. one full storey or more) depending on the full wall height and section depth. The wall capacity should be checked not only at the base of the wall but also at the top of the lap splice. If necessary, an appropriate reduction in moment capacity should be accounted for.

The methodology detailed in Section C5.3.3.2 for columns can be used to assess lap splice performance in walls. In the absence of a more detailed analysis, the equations provided in Section C5.5.3 for columns can be used as indicative values.

**Buckling of vertical reinforcement**

Buckling of reinforcing bars in RC elements is a complex phenomenon and, although the seismic codes contain general detailing requirements to postpone or avoid this, there is currently limited information for assessing existing buildings. Appendix C5G discusses buckling in more detail and recommends an assessment approach.

**C5.5.4 Probable shear strength of walls**

**General approach**

The probable shear strength of the plastic region at the base of a wall, $V_{wall,P}$, can be assessed using a similar approach to that adopted for the columns with some modifications:

$$V_{wall,P} = 0.85 \left( V_c + V_s + V_N \right) \quad \ldots \text{C5.57}$$

The shear strength equation calculates the capacity as the sum of three components:

- $V_c$ = concrete shear-resisting mechanism
- $V_s$ = horizontal reinforcement truss shear-resisting mechanism
- $V_N$ = axial load component.
In more detail:

- the concrete shear resisting mechanism, $V_C$, can be calculated as:

$$V_C = \alpha_P \beta \gamma_p \sqrt{f_c^l} (0.8A_g)$$  \hspace{1cm} \text{...C5.58}

$$\alpha_P = 3 - \frac{M}{Vl_w} \geq 1.0$$  \hspace{1cm} \text{...C5.59}

$$\beta = 0.5 + 20\rho_g \leq 1.0$$  \hspace{1cm} \text{...C5.60}

Figure C5.30 shows the degradation of the shear resisting contribution of concrete as a function of displacement ductility.

- The shear contribution of the effective horizontal reinforcements, $V_s$, is evaluated as follows:

$$V_s = \frac{A_v f_{yh} h_{cr}}{s}$$  \hspace{1cm} \text{...C5.61}

$$h_{cr} = \frac{l'}{\tan\theta_{cr}} \leq h_w$$  \hspace{1cm} \text{...C5.62}

$$l' = l_w - c - c_0$$  \hspace{1cm} \text{...C5.63}

$$\theta_{cr} = 45 - 7.5 \left(\frac{M}{Vl_w} \right) \geq 30^\circ$$  \hspace{1cm} \text{...C5.64}

- The axial load contribution to shear resistance, $V_N$, or is given by:

For a cantilever shear wall:

$$V_N = \frac{l_w - c}{2h_w} P$$  \hspace{1cm} \text{...C5.65}
For a shear wall in double curvature:

\[ V_N = \frac{l_w - c}{h_w} \rho \]  
\[ \ldots \text{C5.66} \]

where:

- \( \rho \) = the ratio of total longitudinal reinforcement over the gross cross-sectional area of the member
- \( A_g \) = gross area of section
- \( A_v \) = horizontal shear reinforcement
- \( f_yh \) = yield strength of transverse reinforcement
- \( s \) = centre-to-centre spacing of shear reinforcement along member
- \( h_w \) = wall height
- \( c \) = the depth of the compression zone
- \( c_0 \) = the cover to the longitudinal bars
- \( l_w \) = wall length

**Note:**
The formulation of shear capacity for walls herein reported has been proposed by Krolicki et al. (2011) and is based on the modified UCSD (University of California, San Diego) shear model proposed by Kowalsky and Priestley (2000) and updated by Priestley et al. (2007), also adopted in Section C5.5.3.3 for the evaluation of the shear capacity of columns.

### C5.5.5.5 Deformation (curvature, rotation and displacement/drift) capacities of shear walls

#### Yield curvature

The yield curvature of RC shear walls can be calculated following the proposed formulation by Priestley et al. (2007), in the same way as outlined in previous sections for beams and columns.

For rectangular shear walls:

\[ \phi_y = \frac{2\epsilon_y}{l_w} \]  
\[ \ldots \text{C5.67} \]

For flanged shear walls:

\[ \phi_y = \frac{1.5\epsilon_y}{l_w} \]  
\[ \ldots \text{C5.68} \]

where:

- \( l_w \) = wall length

#### Ultimate curvature

In general terms, the evaluation of ultimate curvature for shear can be carried out in a similar manner to that presented for columns. Special care should be taken in relation to the particular mechanisms of wall elements.
Note:
The main hypothesis of ‘plane sections remain plane’, i.e. linear strain profile along the wall section length, $l_w$, might not be valid at ULS due to higher concentration of strains in both tension and compression area. Therefore, a traditional section analysis approach may lead to unconservative results and overestimate the curvature/rotation/displacement demand of walls.

However, while acknowledging the limitations of section analysis, it can still be a valuable approach to determine an upper bound of the deformation capacity of an existing wall under an ideal flexurally dominated behaviour.

Interaction with shear (either before or after yielding), local bar buckling or out-of-plane (lateral global) instability can lead to premature failure or achievement of ULS. More information on these failure mechanisms are described in the following sections and in Appendix C5G.

**Plastic hinge length, $L_p$**

As suggested by Priestley et al. (2007), the plastic hinge length of shear walls is more likely to be influenced by tension shift effects than is the case with beams or columns.

Therefore, when compared to the expression for plastic hinge length in beams and columns, an additional term in the plastic hinge equation should be included as a function of the wall length as follows:

$$L_p = k \cdot L_C + 0.1l_w + L_{SP}$$  \(\ldots\text{C5.69}\)

$$k = 0.2 \left( \frac{f_u}{f_y} - 1 \right) \leq 0.08$$  \(\ldots\text{C5.70}\)

$$L_{SP} = 0.022f_yd_b$$  \(\ldots\text{C5.71}\)

where:

$L_C = \text{distance from the critical section to the point of the contraflexure}$

$l_w = \text{wall length}$

Note:
As noted in Section C5.5.3 for columns, the values presented above are typically based on experimental results with reference to relatively well detailed plastic hinge regions and use of deformed bars.

However, as observed following the Canterbury earthquake sequence (Kam, Pampanin and Elwood, 2011; Structural Engineering Society of New Zealand (SESOC) 2011; Sritharan & al., 2014), when dealing with older construction practice, and in the specific case of walls, with:

- low longitudinal reinforcement ratio – i.e. lightly reinforced walls
- construction (cold) joints
- high tensile strength of concrete, and, possibly
- plain round bars,
the plastic hinge length may be concentrated in a very short region with mostly a single main flexural crack, as opposed to distributed cracking over a length. This concentration of tensile inelastic strain demand in the reinforcement resulted in premature fracture of vertical reinforcement.

In fact, while primary cracks occur as a result of the global flexural action on the wall, if low vertical reinforcement ratio is provided the tension force generated by the reinforcing steel – and thus the tensile stress generated in the surrounding concrete – may be insufficient to develop secondary flexural cracks.

Recent studies suggests that even recent design provisions (including NZS 3101:2006 with a specified minimum reinforcement ratio of \( \rho_n \geq \sqrt{f'_c/(4f_y)} \)) may not be sufficient to ensure distributed cracking in the ductile plastic hinge regions, thus potentially resulting in premature bar fracture, and lower-than expected drift capacities (Henry, 2013).

More specifically, not only the total reinforcement ratio along the full section but also the amount (or lack of) longitudinal reinforcement concentrated in the boundary region can facilitate the formation (or impairment) of secondary cracks.

As part of the assessment procedure, such effects should be accounted for in the evaluation of the plastic hinge \( L_p \).

A simple and practical approach would be to assume much smaller values of the plastic hinge length, as \( L_p/5 \), and evaluate its effects on the overall behaviour (limited ductility/deformation capacity).

Also note that large crack openings at the wall base can cause additional problems such as large axial elongations, wall sliding, out-of-plane wall instability.

It is recommended that a plastic hinge length equal to \( L_p/5 \) is adopted (with \( L_p \) derived from the expressions above) in the presence of either:

- plain round bars, or
- low longitudinal reinforcement ratio, i.e. \( \rho_\ell \leq \sqrt{f'_c/(4f_y)} \), or
- inadequately constructed cold joint, e.g. smooth and unroughened interfaces.

### C5.5.5.6 Out-of-plane (lateral) instability

Out-of-plane (or lateral) instability is currently identified as one of the common failure modes of slender rectangular RC walls. This ‘global’ mode of failure, which involves a large portion of a wall element as opposite to the ‘local’ bar buckling phenomenon where a single rebar is affected, was previously observed in experimental studies of rectangular walls. However, it was not considered as a major failure pattern until the recent earthquakes in Chile (2010) and Christchurch (2011). Appendix C5G provides an overview of the issue and a description of current knowledge on the topic.

### C5.5.5.7 Simplified capacity curves for single shear walls

Depending on the failure mode mechanism, force-displacement capacity curves can be derived.

Figures C5.31 and )Shear-Out-of-plane instability (e) Flexure-Out-of-plane instability
Figure C5.32 illustrate the qualitative capacity curves for walls depending on different failure mechanisms as illustrated by Krolicki et al., 2011.

Figure C5.33 presents the flow chart for an assessment procedure for walls as developed and proposed by Krolicki et al., 2011, in line with the component and mechanism based approach adopted throughout these guidelines.
Figure C5.33: Evaluation of wall failure mode (Krolicki et al., 2011)
C5.5.6 Concrete floor diaphragms

C5.5.6.1 General

For most concrete diaphragms the in-plane deformations associated with diaphragm actions will be negligible. Therefore, the assumption of rigid diaphragm behaviour might generally be satisfactory.

One notable exception to this is that stiffness of transfer diaphragms should typically be included explicitly in the analysis (e.g. in the common situation of a suspended ground floor above a basement). In the case of transfer diaphragms, assuming a rigid diaphragm may lead to potentially unrealistically large diaphragm forces.

Note:

When assessing buildings it is important to recognise that there is an inherent difference between the performance and integrity of precast flooring systems and traditional cast-in-situ concrete floors. Precast floors with cast-in-situ concrete topping are not as robust or tolerant to racking movements under earthquake actions as cast-in-situ floors. These will require additional assessment to determine that adequate performance can be achieved.

C5.5.6.2 Diaphragm analysis

Design actions on concrete diaphragms should be determined using a strut and tie analysis.

For buildings that are essentially rectangular, have a relatively uniform distribution of vertical lateral force resisting systems across the plan of the building, and have no significant change of plan with height, simple, hand-drawn strut and tie solutions can be used (refer to Figure C5.34).

![Figure C5.34: Example of a hand-drawn strut and tie solution for simple building (Holmes, 2015)](image-url)
However, buildings with significant asymmetry in the location of lateral force resisting elements (distribution across the building plan, termination up the height of the building, varying stiffness and/or strength between vertical elements) may require a more sophisticated analysis.

For these types of structures, a grillage method can be used to obtain diaphragm design actions (Holmes, 2015). The key steps for this method are as follows and are also shown in Figures C5.35 and C5.36. Further details of the diaphragm grillage modelling methodology are provided in Appendix C5E.

**Step 1**

Determine the geometric properties of the diaphragm elements (i.e. topping thickness, beam sizes, etc) from available structural drawings and site measurements.

**Step 2**

Identify areas of potential diaphragm damage which may limit diaphragm load paths (i.e. floor separation due to beam elongation etc) (refer to Section C5.5.6.3 below).

**Step 3**

Calculate probable capacities of diaphragm collector, tie and strut elements using available structural drawings and site investigation data (refer Section C5.5.6.4).

**Step 4**

Determine grillage section properties and complete the grillage model.

Next, for each principal direction of earthquake loading to be considered complete the following steps.

**Step 5**

Calculate building overstrength factor, $\phi_{ob}$, and overstrength diaphragm inertia forces using the pseudo-Equivalent Static Analysis (pESA) procedure detailed in Section C2.

**Step 6**

Determine ‘floor – forces’, $F_{Di}$, from the pESA and apply these to the nodes in the grillage model associated with vertical lateral load resisting elements.

**Step 7**

Determine vertical element out-of-plane floor forces ‘floor – forces’, $F_{OPi}$, from the pESA and apply these to the nodes in the grillage model.

**Step 8**

Run the grillage model analysis to determine the seismic demands on the diaphragm elements.
Step 9
Check the capacity of the diaphragm elements against the seismic demands.

Step 10
If the diaphragm has enough capacity to resist the seismic demands, go to Step 12. Otherwise, if the seismic demands on selected diaphragm elements exceed their capacity, redistribution can be used to utilise other load paths which may exist.

Step 11
Re-check the capacity of the diaphragm elements against the redistributed building seismic demands. If, after redistribution, the diaphragm does not have adequate capacity to resist the seismic demands then reduce the diaphragm inertia forces and return to Step 6. If the diaphragm has adequate capacity to resist the redistributed seismic demands proceed to Step 12.
STEP 1
- Calculate geometric properties of diaphragm elements

STEP 2
- Identify areas of localised diaphragm damage which may limit diaphragm load paths

STEP 3
- Determine probable capacities of diaphragm collector, tie and strut elements

STEP 4
- Determine grillage section properties
  - For each principal direction:
    - Calculate building overstrength factor, $\phi_{ob}$, and overstrength diaphragm inertia forces, $F_{os,i}$

STEP 5
- Determine floor compatibility forces, $F_{D,i}$

STEP 6
- Determine vertical element out-of-plane floor forces, $F_{OP,i}$

STEP 7
- Determine force demands on diaphragm elements

STEP 8
- Has the strength of diaphragm elements been exceeded?
  - $Y$: Reduce diaphragm inertia forces below $F_{os,i}$
  - $N$: Redistribute diaphragm forces away from yielding elements

STEP 9
- Has the strength of diaphragm elements been exceeded?
  - $Y$: Refer Figure C5.39
  - $N$: Continue with the assessment procedure

Figure C5.35: Summary of diaphragm assessment procedure – Steps 1 to 11
**STEP 12**

From Figure C5.38

**STEP 13**

Calculate drift capacity of diaphragm components

**STEP 14**

Calculate interstorey drift demands from NZS 1170.5

**STEP 15**

Determine %NBS for diaphragm drift capacity

**STEP 16**

Is %NBS for diaphragm strength > %NBS for diaphragm drift capacity

Y

N

**STEP 18**

%NBS for drift capacity governs diaphragm capacity

%NBS for strength governs diaphragm capacity

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**Figure C5.36: Summary of diaphragm assessment procedure – Steps 12 to 17**

**Step 12**

Determine %NBS for the diaphragm in terms of strength (refer to Section C5.5.6.4). If the capacity of the diaphragm is greater than the seismic demands calculated using the building overstrength factor, $\phi_{ob}$, the diaphragm can be taken as 100%NBS. If the diaphragm demands were reduced below the building overstrength demands in Step 11, the %NBS for each diaphragm element should be determined as follows:

$$\%NBS = 100 \times \frac{0.9R_{prob}}{K_d R_{E, \mu=1.25}}$$

..C5.72

where:

- $R_{prob}$ = probable capacity of diaphragm element calculated in Step 3
- $R_{E, \mu=1.25}$ = diaphragm element demand calculated using the pESA procedure detailed in Section C2, with the base shear $V_E$ calculated from Section 6.2 of NZS 1170.5:2004 using $\mu = 1.25$ and $S_p = 0.9$
- $K_d$ = demand-side multiplier such that $K_d = 1.5$ for diaphragm collector elements and $K_d = 1.0$ for all other ties and struts.

Redistribution between diaphragm elements is permitted. The %NBS for the diaphragm in terms of strength is the minimum of the %NBS values assessed for each individual diaphragm element.

**Note:**

A higher demand side multiplier of 1.5 is applicable to collector elements recognising that these elements are force controlled, and typically have low redundancy and a high consequence of failure. The demand side multiplier of 1.5 is intended to provide a margin of resilience.
Step 13

Calculate inter-storey drift capacity, $\theta_{SC}$, of diaphragm components. This includes assessing the precast concrete floor units for loss of support and assessing the seismic capacity of the units themselves (refer to Section C5.5.6.3).

Step 14

Calculate inter-storey drift demands, $\theta_{SD}$, in accordance with Section C2 of these guidelines. Section C5.5.6.5 below provides additional guidance on how the NZS 1170.5:2004 structural performance factor, $S_p$, should be applied.

Step 15

Determine $\%NBS$ for the diaphragm in terms of inter-storey drift. The $\%NBS$ for each diaphragm element should be determined as follows:

$$\%NBS = 100 \frac{\theta_{SC}}{K_d \theta_{SD}}$$

...C5.73

where:

$\theta_{SC}$ = inter-storey drift capacity of diaphragm component

$\theta_{SD}$ = inter-storey drift demand on diaphragm component

$K_d$ = demand-side multiplier such that $K_d = 1.5$ for precast concrete diaphragm elements and their support, and $K_d = 1.0$ for in situ concrete diaphragm elements.

The $\%NBS$ for the diaphragm in terms of inter-storey drift is the minimum of the $\%NBS$ values assessed for each individual diaphragm element.

Step 16

Check if the $\%NBS$ for the diaphragm in terms of strength calculated in Step 12 is greater than the $\%NBS$ for the diaphragm in terms of inter-storey drift calculated in Step 15.

Step 17

The $\%NBS$ for the diaphragm is the minimum of the two $\%NBS$ values considered in Step 16.

C5.5.6.3 Diaphragm damage due to deformation compatibility

Deformation demands of the primary lateral force resisting systems can cause damage to the diaphragm structure (as a result of beam elongation or incompatible relative displacements between the floor and adjacent beams, walls or steel braced frames). Figure C5.37 illustrates an example of diaphragm damage due to beam elongation.

The assessment of inter-storey drift capacity of diaphragms consisting of precast concrete components needs to consider the following:

- loss of support of precast floor units, and
- failure of precast floor units due to seismic actions, including the consideration of incompatible displacements.
Appendix C5F provides an assessment procedure for precast floors with cast-in-situ concrete topping.

**Note:**
Precast floors with cast-in-situ concrete topping are not as robust or tolerant to racking movements as traditional cast-in-situ concrete floors. Failure of a precast floor unit in the upper level of a building is likely to result in progressive collapse of all floors below that level. Therefore, additional assessment is recommended to ensure that adequate performance can be achieved during an earthquake.

### C5.5.6.4 Assessment of diaphragm capacities

The capacity of diaphragm strut and tie elements can be calculated in accordance with Appendix A of NZS 3101:2006 (SNZ, 2006) using probable material strengths and a strength reduction factor, $\phi$, equal to 1.0. Reduction factors $\beta_n$ and $\beta_s$ should be taken as specified in NZS 3101:2006.

### C5.5.6.5 Inter-storey drift demands on diaphragm components

Inter-storey drift demands on diaphragm components can be determined in accordance with one of the applicable analysis methods detailed in Section C2 except as modified below (Fenwick et al., 2010):

- When calculating member elongations the structural performance factor, $S_p$, adopted for the primary lateral resisting system can be used to determine the plastic hinge rotations.
- When assessing brittle failure modes of precast concrete components (i.e. web-splitting of hollow core floor units, loss of support, etc) the peak displacements determined from the analysis of the primary lateral load resisting system should be increased by $1/S_p$, where the value of $S_p$ is that used in the analysis of the primary lateral load resisting system.
C5.6 Global Capacity of Moment Resisting Concrete Frame Buildings

C5.6.1 Evaluation of the hierarchy of strength and sequence of events for a beam-column joint subassembly

Once the flexural and shear capacity of the components are evaluated, the hierarchy of strength and expected sequence of events within a beam-column joint can be carried out by comparing capacity and demand curves within an M-N (moment-axial load) performance domain.

Figure C5.38 illustrates an example of the M-N performance domain adopted to predict the sequence of events and the level of damage in the joint panel zone of a 2D exterior beam-column joint subassembly. According to such a procedure, the capacities of beams, columns and joints need to be evaluated in terms of a common parameter. This is recommended to be the equivalent moment in the column, based on equilibrium considerations corresponding to the selected limit state (e.g. cracking/“yielding” or peak capacity in the joint versus yielding of beams and columns).

The order and “distance” of the events (e.g. beam hinging, joint shear, column hinging) can also strongly depend on the axial load demand. If a constant axial load was assumed, as is often done for simplicity, an erroneous sequence of events might be predicted leading to the potential implementation of an incorrect retrofit strategy.

Note:
In the case of the exterior joint shown as an example in Figure C5.38, a shear hinge mechanism with extensive damage of the joint before any hinging of beams or columns was expected and predicted, using a proper demand curve (refer to the table in Figure C5.38) and later confirmed by the experimental tests.

However, as anticipated, the order and “distance” of the events strongly depend on the assumption on the axial load demand curve.

If a constant axial load curve is used (in this case $N = -100$ kN as shown in Figure C5.38), as is often selected in experimental tests and analytical assessment methodology, only a relatively small increase in the joint strengthening would appear necessary for the retrofit intervention.

However, in reality such a strengthening solution would lead to the formation of a column hinging before any beam hinging. This would possibly result in the development of a soft-storey mechanism in spite of the (generally quite expensive and invasive) retrofit intervention already implemented.
Figure C5.38: Example of evaluation of hierarchy of strengths and sequence of events: moment-axial load, M-N, performance domain for an exterior beam-column joint in as-built configuration, (after Pampanin et al., 2007)

Note:

Figure C5.39 and Table C5.12 show further examples of hierarchy of strength evaluation within an M-N interaction diagram for interior and exterior beam-column joints belonging to the Red Book Case Study Building (Brundson and Bull, XXXX).

The case study building consist of a 10 storey reinforced concrete building, designed according to NZS 3101:1995 with moment-resisting frames in both directions.

The results of the hierarchy of strength evaluation for one exterior and one interior beam column joints, belonging to the 5th floor, are herein shown on the left-hand and right-hand side of Figure C5.39 and Table C5.12. More details can be found in Taslignedik et al. 2016.

Figure C5.39: Evaluation of hierarchy of strengths and sequence of events: Moment-Axial Load, M-N, Performance-Domain for an exterior beam-column joint in as-built configuration (after Taslidedik et al., 2014, 2015)
### Table C5.12: Sequence of events evaluated for the beam-column joints of Figure C5.39

<table>
<thead>
<tr>
<th>External RC beam-column joints</th>
<th>Internal RC beam-column joints</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Member</strong></td>
<td><strong>Failure</strong></td>
</tr>
<tr>
<td>Beams (-1 &amp; +1)</td>
<td>Flexural hinging</td>
</tr>
<tr>
<td>Joint on tension side (-2)</td>
<td>Shear failure</td>
</tr>
<tr>
<td>Column on tension side (-3)</td>
<td>Flexural hinging</td>
</tr>
<tr>
<td>Joint on compression side (+2)</td>
<td>Shear failure</td>
</tr>
<tr>
<td>Column on compression side (+3)</td>
<td>Flexural hinging</td>
</tr>
</tbody>
</table>

### C5.6.2 Effect of varying axial load on joint capacity

The capacity of a beam-column joint, particularly when characterised by poor detailing and lack of transverse reinforcement as typically found in older buildings, is strongly affected by the variation of the axial load. This was anticipated above when introducing principal stresses instead of nominal shear stress as a more realistic damage indicator.

Therefore, appropriate demand curves for beam-column joint systems should account for the variation of axial load due to the lateral sway mechanism, for either opening and closing of the joint (Figure C5.40). Otherwise, incorrect and non-conservative assessment of the sequence of events can result and lead to inadequate – and not necessarily conservative – design of any retrofit intervention.

![Variation of axial load due to frame sway mechanism and its effects on the hierarchy of strength of beam-column joint subassemblies](image_url)
Note:

Most of the experimental cyclic tests on joint subassemblies (as well as column-to-foundation connections) are carried out, for simplicity, under a constant axial load regime in the column/joint.

While this simplified testing procedure is not expected to have a substantial effect on the behaviour of well-designed specimens, in the case of poorly detailed subassemblies the effect on damage level and mechanisms could be significant.

In general, the axial load on a column can be expressed as:

\[ N = N_g \pm F \alpha \]  \hspace{1cm} \text{\ldots C5.74}

where:

- \( N_g \) = the axial load due to gravity load
- \( F \) = the lateral force (base shear capacity)
- \( \alpha \) = depends on the global geometry of the building (height and total bay length, \( L \), as shown in Figure C5.41)

Such variation of axial load due to the seismic action can be substantial for exterior beam-column joints. It can be 30-50% or higher, with further increase when considering bidirectional loading.

On the other hand, as a first approximation (especially if there are only two or three bays) the variation of axial load in interior beam-column joints can either be neglected or assumed to be in the order of 10-20%.

![Figure C5.41: Example of evaluation of variation of axial load in a frame](image)
C5.6.3 Upper and lower bounds of base shear capacity and force-displacement curves

C5.6.3.1 General

Once the hierarchy of strength and sequence of events of all the beam-column joint subassemblies within a frame have been evaluated, the global mechanism of the frame can be analysed.

In general, as shown in Table C5.13, upper and lower bounds of the lateral load capacity (i.e. base shear or overturning moment) will be given by a soft-storey mechanism and a beam sway mechanism respectively. Any mixed sidesway mechanisms, including possible shear hinging in the joint, would provide an in-between capacity curve.

Note:
The overall Overturning Moment (OTM) in a frame is given by the sum of the moments at the column bases and the contribution of the axial load variation in the columns “collected” from the shear contribution of the beam. Therefore, each mixed mechanism can be evaluated by estimating the moment in each beam resulting from the equilibrium of the subassembly, as follows:

\[ OTM = \sum_i M_{coli} + \left( \sum_{x} V_{end\ beam,x} \right) L \] …C5.75

Table C5.13: Upper and lower bounds of frame capacity due to column and beam sway mechanisms, and in-between capacity due to mixed sway mechanism

<table>
<thead>
<tr>
<th>Upper bound</th>
<th>Lower bound</th>
<th>In between</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam sidesway mechanism</td>
<td>Column sidesway mechanism</td>
<td>Mixed sidesway mechanism</td>
</tr>
<tr>
<td><img src="image1" alt="Beam Sidesway Mechanism" /></td>
<td><img src="image2" alt="Column Sidesway Mechanism" /></td>
<td><img src="image3" alt="Mixed Sidesway Mechanism" /></td>
</tr>
</tbody>
</table>

\[ OTM, 1 = \sum_i M_{coli} + \left( \sum_{n} V_{end\ beam,n} \right) L \]
\[ V_{b,1} = \frac{ OTM }{ H_{eff, beam \ sidesway} } \]

\[ OTM, 2 = \sum_i M_{coli} \]
\[ V_{b,2} = \frac{ OTM }{ H_{eff, col \ sidesway} } \]

\[ OTM = \sum_i M_{coli} + \left( \sum_{x} V_{end\ beam,x} \right) L \]
\[ V_{b,3} = \frac{ OTM }{ H_{eff, mixed sidesway} } \]

Note:
\[ \sum_i M_{coli} = \text{sum of base column moments} \]
\[ \sum_{n} V_{end\ beam,n} = \text{sum of end beam shears for all n levels} \]
\[ L = \text{frame full span} \]
C5.6.3.2 Beam sidesway mechanism

\[ OTM, 1 = V_{b,1} \cdot H_{eff} = \sum_i M_{coli} + (\sum_n V_{\text{end beam},n}) L \]  \( \ldots \text{C5.76} \)

where:

\[ V_{\text{end beam}} = \text{the additional column axial load due to the beam shear (evaluated as corresponding to maximum flexural capacity).} \]

This provides an upper bound of the lateral load resistance capacity.

C5.6.3.3 Column sway mechanism

\[ OTM, 2 = \sum_i M_{coli} = V_{b,2} \cdot 0.5h \]  \( \ldots \text{C5.77} \)

where:

\[ \sum_i M_{coli} = \text{Sum of Moment of the columns at the base} \]
\[ 0.5h = \text{point of contraflexure of one floor} \]

This provides a lower bound of the lateral load resistance capacity.

C5.6.3.4 Mixed mechanism

\[ OTM, 3 = V_{b,3} \cdot H_{eff} = \sum_i M_{coli} + (\sum_x V^*_{\text{end beam},x}) L \]  \( \ldots \text{C5.78} \)

where:

\[ V^*_{\text{end beam}} \text{ is determined from the minimum value (expressed as equivalent beam moment) between the beam flexural capacities, joint equivalent moments, column flexural capacities, and column shear capacities, depending on strength hierarchy at local level.} \]

This base shear value, corresponding to a mixed mechanism, \( V_{b,3} \), should be in between the upper and lower bound determined from a beam sway, \( V_{b,1} \), and a column sway, \( V_{b,2} \), mechanisms, respectively.

When combining the information on yielding and ultimate (limit states) drift displacement of the frame corresponding to the most critical mechanism, the global force-displacement curve of this frame can be evaluated as shown in Figure C5.42.

The structure’s performance can thus be assessed against any given level of earthquake intensity, using an Acceleration Displacement Response Spectrum (ADRS) approach as described in Section C2.
Global Capacity of Wall Buildings

C5.7.1 General

The assessment of the overall behaviour of a building’s structural system in which seismic resistance has been assigned to reinforced concrete structural walls will probably be less elaborate than that for frame systems.

In the presence of robust walls, the contribution to seismic resistance of other elements with a primary role of supporting gravity loads may often be, at a first stage, neglected. The detailing of such frame components only needs checking to satisfy any displacement compatibility issues with the overall 3D response (including torsion) of the building system.

In such cases, it is important to check the displacement-drift capacity of non-ductile columns for displacement demand higher than that corresponding to the ULS displacement capacity of the main wall-lateral resisting system (refer to Section C1 for details of this Critical Structural Weakness).

The presence of alternative load paths and overall redundancy characteristics should be checked in order to avoid progressive and catastrophic collapse, as observed in the CTV building after the 22 February 2011 Christchurch earthquake.
Note:
If the contribution of such frame systems to seismic capacity is judged to be more significant or the system needs to rely on their seismic contribution to satisfy seismic performance criteria, the building should be treated as a dual frame-wall building and assessed as outlined in Section C5.8.

### C5.7.2 Evaluation approach

The first step is to evaluate the total force-displacement capacity curve of the wall system in each orthogonal direction (i.e. assuming 2D response) as the sum in parallel of all walls contributing in that direction. This is shown in Figure C5.44 with reference to the layout of a wall system shown in Figure C5.43.

**Figure C5.43:** (Elastically calculated) torsional effects in a walled building

**Figure C5.44:** Bilinear idealisation of ductile element and system response for a wall building shown in Figure C5.43
Figure C5.44 shows the global capacity curve and the individual contribution of each wall system.

The relationship between ductilities developed in walls withdifferent dimensions and that of the wall system as a whole can be appreciated.

As the wall with greatest length will yield first, it is likely that, assuming a flexurally dominated behaviour, the associated displacement capacity of such walls will govern the overall displacement capacity of the system. However, other brittle mechanisms can occur first on individual walls and should be carefully checked.

This procedure is based on the use of a simplified analytical approach where the two orthogonal directions are, at a first stage, considered to be decoupled.

This approximation is more appropriate when dealing with rectangular walls and acceptable as a first step, when considering C-shape or T-shape walls with poor connection details in the corner/regions.

When good connection between web and flange are present in T- or C-shaped walls, the actual behaviour of the walls in both longitudinal and trasverse directions should be evaluated.

In any case, the 3D response effects should then be also accounted for. These include, for example:

- slab coupling effects between walls oriented orthogonally but close to each other, and
- possible response amplifications to the displacement/ductility demand due to inelastic torsional effects (refer to Section C2 for details of procedures to account for inelastic torsional effects).

**C5.8 Global Capacity of Dual Frame-Wall Concrete Buildings**

**C5.8.1 General**

In dual systems, elements resisting lateral forces in a given direction of the building may have significantly different behaviour characteristics. Mechanisms associated with their ductile response may also be very different. Typical examples are buildings where lateral forces in different parallel vertical planes are resisted by either ductile frames or ductile walls. Walls forming a service core over the full height of the building are common. They may be assigned to resist a major part of the lateral forces, while primarily gravity load carrying frames may also be required to provide a significant fraction of the required seismic strength.

Irrespective of whether elastic or post-yield behaviour is considered, displacement compatibility requirements (Paulay and Priestley, 1992) over the full height of the building need to be considered. The presence of a rigid diaphragm, with an ability to transfer significant in-plane dynamically induced floor forces to the different vertical elements, is a prerequisite. Therefore, the examination of diaphragm-wall connections is particularly important (refer to Section C5.5.6 for more details).
During the ductile dynamic response of such dual systems, very different displacement ductility demands may arise for each of the two types of individual lateral resisting system. One purpose of the assessment procedure is to identify the element with the smallest displacement capacity. Wall elements, often representing significant fractions of the probable lateral strength of the system, are typical examples. They control the displacement capacity of the system.

Major advantages of such dual systems are that displacement ductilities imposed on frames are generally very moderate, and that dynamic displacement demands are not sensitive to modal effects, as in the case of frame systems. Moreover, in comparison with frame (-only) or wall(-only) systems, dual systems provide superior drift control. Provided that potential plastic hinges are detailed for moderate curvature ductility demands, column sway mechanisms in any storey of the frames are acceptable.

The assessment procedure outlined is applicable to any combination of walls and frames, provided that no gross vertical irregularities, such as discontinuities in walls, exist. It is based on displacement-focused or displacement-based treatment of ductile reinforced concrete systems introduced in Paulay and Restrepo (1998); Paulay (2000, 2001b and 2002) and on a redefinition of strength-dependent component stiffness (Paulay, 2001a).

**Note:**

For more recent information on displacement-based design for dual systems that can be used for the assessment procedure Sullivan et al., 2012.

This enables the same assessment procedure to be carried out for strength and displacement-based performance criteria. The displacement ductility capacity of a dual system needs to be made dependent on the displacement capacity of its critical element.
**C5.8.2 Derivation of global force-displacement capacity curve**

**C5.8.2.1 Assessment approach**

As the walls are expected to govern the behaviour of the dual system, both in terms of strength and stiffness, it is recommended to start the assessment of a dual system from the assessment of the wall system(s).

In fact, because the wall remain essentially elastic above the plastic region at the base during ductile system response, their deformations will control that of the overall system. Moreover, in general, the displacement capacity of the walls, rather than that of the frames, should be expected to control the performance limit state.

Hence, wall displacement capacity should be estimated and compared with the corresponding displacement ductility demands generated in the frames.

**C5.8.2.2 Step by step procedure**

**Step 1 Estimate the post-elastic mechanism of walls and their contribution to lateral force resistance**

The nonlinear mechanism of the walls of a dual system is expected to comprise plastic hinges at the base of each wall. A detailed study of the wall capacity along the height, as outlined in Section C5.5.5, is required to verify this.

Based on the procedure presented in this section for single cantilever walls, moment-curvature analyses of the wall cross-sections can be computed at each level accounting for the axial load variation and change in longitudinal and transverse reinforcements. The wall flexural strength should be checked against the shear strength to detect premature shear failure along the wall height. This failure is likely to govern the behaviour of walls more than columns.

As shown by the dash/dot line in Figure C5.46, the moment capacity gradually reduces along the height as a consequence of the reduced axial load and longitudinal reinforcement amount.

![Figure C5.46: Displacement response of a wall structure (Priestley et al., 2007)](image-url)
Assuming a typical first-mode distribution of lateral forces (i.e. inverted triangular), determine the distribution of the bending moment up the wall height corresponding to the wall-base flexural strength (the solid line in Figure C5.46(b)).

Determine the extent of the wall region over which the shear stress is such that diagonal cracking is to be expected. Over this region, tension shift effects resulting from diagonal cracking will increase the apparent moment. This influence can be reasonably represented by shifting the moment profile over the affected region up by a distance equal to half the wall length, $l_w/2$ (dashed line in Figure C5.46(b)).

The critical section of the wall can be identified comparing the capacity and demand moment envelope (dash/dot and dashed line in Figure C5.46(b)). If the capacity exceeds the demand at all the levels above the base, such as in the example in Figure C5.46(b), the inelastic response can be assumed as concentrated at the base only. Otherwise, plastic hinging is expected at the level where the demand is higher than the capacity.

Characterise the push-over curve of the single-degree-of-freedom (SDOF) system assuming, in first instance, a cantilever wall scheme with $H = H_{eff}$.

Based on the probable strength of the examined sections of all walls of the system, quantify the total overturning moment that can be carried by these walls, $M_{w,b}$ (subsequently referred to as the wall element).

With this evaluation of the overturning moment capacity of the wall element, $M_{w,b}$, (refer to Figure C5.48(a)), its probable base shear strength can be estimated from:

$$\sum V_{wp} = M_{w,b}/H_{eff} \quad \ldots\text{C5.79}$$

The effective height of the wall element, $H_{eff}$, is given by the approximate position of its point of contraflexure Figure C5.48a. As a first approximation it can be assumed that $H_{eff} = 0.67H_w$.

When a more slender wall element is used, its probable base strength will be smaller and the point of zero wall moment will be at a lower level, resulting in $H_{eff} < 0.67H_w$.

While the storey shear strength provided by the frames can be evaluated with a relatively high degree of precision, the likely shear demand on the walls is less certain. This is because walls are significantly more sensitive to differences between estimated and real seismic demands.

Therefore, comparisons of probable wall storey shear strength should be conducted with caution as these are largely dependent on the horizontal shear reinforcement which has been provided.

The displacement capacity at the yielding and ultimate limit state conditions can be computed according to Section C5.5.5.
Step 2  Establish the post-elastic mechanism of frames and their contribution to lateral force resistance

Following the procedure outlined in Section C5.5 the probable strength of beams, column and joints are evaluated as well as the hierarchy of strength of column/beam/joint and the overall probable mechanism.

The contribution of the frame members at each floor can therefore be computed imposing the drift corresponding to the yielding and ultimate limit state in the wall on the weaker frame, as illustrated in Figure C5.47.

This allows the computation of the distribution of bending moment, shear and axial load on the frames and the corresponding actions transmitted to the wall.

To obtain a more refined assessment of the wall behaviour and failure mode, the shear and flexural strength previously calculated in Step 1 can be now compared with a more refined estimation of the shear and bending moment demand determined accounting for the contribution of the frames at each floor.

Note:

Figures C5.47 and C5.48 illustrate the procedure described at Step 2, with a kinematically admissible sway mechanism. Plastic hinges introduce a total moment of $\sum M_{pi}$ to the four (equivalent) columns at the level of the beams. This is proportional to the storey shear force, $V_{pi}$. Note that the overturning moments transmitted from storeys above by means of axial forces in the columns are not shown here.

These figures also illustrate the stepwise estimation of the contribution to total probable overturning moment capacity and storey shear force of both the frames and the walls.
Part C – Detailed Seismic Assessment

C5: Concrete Buildings

Draft Version 2016_C – 10/10/2016 NZ1-9503830-62 0.62

(a) Normalised overturning moments ($M/hV_b$)
(b) Normalised storey shears ($V_s/V_b$)
(c) Displacement profiles

Figure C5.48: Stepwise estimation of the contribution of a frame and a wall element to probable lateral strength and corresponding displacements of a dual system

Step 3: Determine the stiffness and displacement capacity of dual systems

Once the strength contribution of frame members at specific levels of drift has been assessed, the base shear contribution of the frame, wall and resultant dual system can be computed by dividing the total overturning moment by the effective height, $H_{eff}$, as suggested in the previous Step 1. In the case of dual systems, the effective height of the frame can be assumed to be equal to the effective height of the wall.

Alternatively, and more practically, the base shear of the dual system can be obtained by:

- summing directly (in parallel, thus assuming equal displacement) the pushover curves of the SDOFs of the wall and the frames, or
- estimating the OTM of the dual system considering the contribution of wall and frame elements (refer to Figure C5.47b).

Note:

Figure C5.47(b) presents the overall simplified (bilinear modelling) force-displacement capacity curve of the dual system, summarising the procedure discussed in Step 3 and is similar to that shown in Figure C2.9 of Section 2.5.11 on Mixed Ductility Systems. Note that this figure represents the expected behaviour of the schematic dual system shown in Figure C5.46 (i.e. a dual system comprising of a central wall and beams coupling to two external columns) as specific assumptions were made to illustrate the simple details of these calculations.

As Figure C5.48(b) shows, an approximately equal contribution (50-50) to the probable base shear strength of the dual system, $V_{dual,p}$, was found to be provided by the wall and the frame elements.
The relative nominal yield displacements at level $H_e$, were found to be:

- $\Delta_{wy} = 1.00$ displacement units for the wall element, and
- $\Delta_{fy} = 1.72$ displacement units for the frame element.

Therefore, the normalised stiffness of the wall and frame elements are, respectively:

$$k_w = \frac{V_{wp}}{\Delta_{wy}} = \frac{0.5}{1.0} = 0.5 \quad \ldots \text{C5.80}$$

$$k_f = \frac{V_{fp}}{\Delta_{fy}} = \frac{0.5}{1.72} = 0.29 \quad \ldots \text{C5.81}$$

Hence the relative nominal yield displacement of the dual system is:

$$\Delta_y = \frac{V_{\text{dual,p}}}{(K_w + K_f)} = \frac{1.00}{(0.5 + 0.29)} = 1.27 \text{ displacement units}$$

The bilinear idealisation of the force-displacement curve for frame, wall and dual system behaviour, shown in Figure C5.48(b), confirms these quantities.

### C5.9 Improving the Seismic Performance of Concrete Buildings

Alternative seismic retrofit and strengthening solutions for concrete buildings have been studied and adopted in practical applications ranging from conventional techniques (e.g. using braces, walls, jacketing or infills) to more recent approaches including base isolation, supplemental damping devices or involving advanced materials such as fibre reinforced polymers (FRPs) and shape memory alloys (SMAs). Refer to international guidelines such as fib, (2003), EC8-part 3 (2003) FEMA 547 (XX); ASCE-41-13 (2014).

Most of these retrofit techniques have evolved into viable upgrades. However, issues of cost, invasiveness, architectural aesthetics, heritage protection and practical implementation still remain the most challenging aspects of any intervention.

Based on lessons learned from recent major earthquakes and on extensive experimental and analytical data, it is increasingly evident that major – and sometimes controversial – issues can arise in, for example:

- deciding whether the retrofit is actually needed and, if so, in what proportions and to what extent
- assessing and predicting the expected seismic response pre- and post-intervention by relying upon alternative analytical/numerical tools and methods
- evaluating the effects of the presence of infills, partitions or general “non-structural” elements on the seismic response of the overall structure, which is more typically and improperly evaluated considering only the “skeleton”
- deciding, counter-intuitively, to “weaken” one or more structural components in order to “strengthen” the whole structure
- adopting a selective upgrading to independently modify strength, stiffness or ductility capacity
• relying upon the deformation capacity of an under-designed member to comply with the
displacement compatibility issues imposed by the overall structure, and/or
• defining a desired or acceptable level of damage that the retrofit structure should sustain
after a given seismic event: i.e. targeting a specific performance level after the retrofit.

Regardless of what technical solution is adopted, the efficiency of a retrofit strategy on a
reinforced concrete building depends strongly on a proper assessment of the internal
hierarchy of strength as well as on the expected sequence of events and damage/failure
mechanisms within:
• a frame system (i.e. shear damage and failure in the joint region, flexural hinging or shear
failure in beam and column elements, or
• a wall system (i.e. sliding, flexural or shear failure, lateral instability etc), or
• a combination of these (dual system).

Following a conceptually similar procedure included in these guidelines, and in particular
the SLaMA method, the overall lateral force vs. displacement curve of the building system
can be computed before and after alternative retrofit interventions and the performance point
of the structure under different earthquake intensity computed, including the new level of
\%NBS achievable when improving the behaviour of individual elements.

This approach allows to gain a direct appreciation of the incremental benefits achievable
when implementing specific retrofit interventions or combination of them.

The retrofit strategy can follow a selective intervention, i.e. strength-only, ductility-only,
stiffness-only, as well as selective weakening, or a combination of the above.

An overview of alternative performance-based retrofit strategies and technical solutions for
Reinforced Concrete buildings, developed and/or refined in the past decade few years as part
of the multi-year research project “Retrofit Solutions for NZ multi-storey Buildings”, funded
by the FRST (Foundation of Research Science and Technology from 2004-2010) can be
found in Pampanin, 2009. Pampanin et al., 2010).

References

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Appendix C5A: Key Milestones in the Evolution of New Zealand Concrete Design Standards, and Historical Concrete Property Requirements and Design Specifications in New Zealand

Table C5A.1: Summary of key milestones in the evolution of New Zealand concrete design standards (modified after Fenwick and MacRae, 2011)

<table>
<thead>
<tr>
<th>Period</th>
<th>Loading Standard</th>
<th>Concrete Standard</th>
<th>Major changes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-1957</td>
<td>1935 Model Bylaws</td>
<td>No seismic provisions</td>
<td>While there were no specific seismic requirements, 135 degree hooks were already shown for stirrups in RC construction (clause 409). Maximum spacing of stirrups was 2/3 of the internal lever arm (clause 616). Development of plain round longitudinal bars was often by 180 degree hooks.</td>
</tr>
<tr>
<td>1957-1964</td>
<td>NZSS 95 - Pt IV Basic Loads to be used and methods of application (1955)</td>
<td>UK concrete Code of Practice, CP114:1957 (No seismic provisions) and NZSS 95, Pt V (1939)</td>
<td>Section properties of members were permitted to be based on gross sections, transformed un-cracked sections, or transformed cracked sections (Fenwick and MacRae, 2009).</td>
</tr>
<tr>
<td>1964-1968/71</td>
<td>NZSS 1900 Basic Design Loads Chapter 8 (1964)</td>
<td>Design and Construction, Concrete, Chapter 9.3, 1964 (No seismic provisions)</td>
<td>Essentially, no seismic details were specified. It is likely that reinforcement was inadequately anchored for seismic actions, particularly in columns. Plain round bars were used extensively during this period.</td>
</tr>
<tr>
<td>1982</td>
<td>NZ4203:1976</td>
<td>ACI 318:1971 or provisional NZ Concrete Standard, NZS 3101:1970</td>
<td>Ultimate Strength Design used Strength Reduction Factors of 0.9 for beams, 0.75 for confined columns and 0.7 for unconfined columns. Member stiffness for seismic analysis recommended as 75% gross section stiffness. Provisions for detailing potential plastic hinge regions introduced:  - some shear reinforcement to resist the gravity induced shear and the shear corresponding to flexural strength in the potential plastic hinge region  - lapping of bars in specified potential plastic hinge regions not permitted  - some column confinement required where axial load ratio bigger than 40% (N_b) (balanced condition).</td>
</tr>
</tbody>
</table>
### Part C – Detailed Seismic Assessment

#### C5: Concrete Buildings

<table>
<thead>
<tr>
<th>Period</th>
<th>Loading Standard</th>
<th>Concrete Standard</th>
<th>Major changes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1982-1995</td>
<td>NZS 4203:1984</td>
<td>NZS 3101:1982</td>
<td>Capacity design required to ensure sum of column strengths greater than the sum of beam strengths (with no minimum ratio). Modifications to strength reduction factors: 0.9 for flexure in beams and confined columns; 0.7 for unconfined column with axial load higher than $0.1A_f f'$; and 0.9 for zero axial load (clause 4.3.1). Member stiffness 0.5 times the gross section stiffness for beams and 1.0 for columns (clause C3.5.5.1).</td>
</tr>
</tbody>
</table>

**Detailing**

- Confinement of all potential column plastic hinges required, depending on the maximum design axial load level in the column due to the gravity and earthquake actions (clause 6.5.4.3). It was greater than in the previous standards.
- Lapped bars not permitted at floor levels in columns where there was a possibility of yielding.
- Shear reinforcement requirements in plastic hinge zones more conservative.
- Specific anti-buckling bars in potential plastic hinge regions.
- Joint shear reinforcement development requirements and reinforcing increased.
- Column ties anchored by 135 degrees in cover concrete.
- Beam bars in external joints likely to be bent away from the joint core.
- Columns not designed for earthquake with $\phi=0.7$ were permitted to have 6 mm reinforcement at spacing no greater than (i) the minimum column cross sectional dimension, (ii) 16 times the longitudinal diameter.

**Capacity design**

Capacity design requirements:

- Over-strength moments in beams were taken as 1.25 or 1.4 times the ideal flexural strength of beams with grade 275 and 380 steel respectively (clause C3.5.1.3).
- Design for a Strong Column Weak-Beam frame mechanism was specified in the commentary (refer to NZS 3101:1982, Appendix C3A). This encouraged potential primary plastic regions to be in the beams, except at the column bases. To obtain the column design actions for flexure, shear and axial force, this included considering:
  - the maximum beam over-strength moments that could be applied to a joint which affected the corresponding static column demands
  - changes in distribution of column moments due to higher elastic and inelastic mode behaviour, with a dynamic magnification factor
  - bi-axial moments on columns which were part of two orthogonal frames, and
  - effects of beams yielding simultaneously over the frame.
The required minimum ratio of the sum of the nominal column flexural strengths to the sum of the nominal beam flexural strengths at beam-column joint centreline in one way frames ranged from 1.6 to 2.4. In many cases the minimum ratios were exceeded as the flexural strengths of the column changed between the top and bottom of the joint zone; and for practical purposes the same longitudinal reinforcement was used in the column on each side of the joint zone.

This method of designing columns for seismic actions was adopted into NZS 3101:1995 and retained with minor modifications in NZS 3101:2006.

An effective width of floor slab (usually 2 to 4 times the depth of the slab measured from the column faces) was assumed to contribute to beam overstrength (clause 6.5.3.2 (e)), which was smaller than that in later standards.

Diaphragm Design (refer to Section 10.5.6).

Floors are designed for the smaller of the maximum forces that could be resisted by the lateral force system, or for the forces from the “parts and portions” section of the loadings standard.

Nominal requirements were given for reinforcement to tie the floor into the building and for the use of precast flooring elements.

<table>
<thead>
<tr>
<th>Period</th>
<th>Loading Standard</th>
<th>Concrete Standard</th>
<th>Major changes</th>
</tr>
</thead>
</table>
| 1995-2006   | NZS 4203:1992    | NZS 3101:1995    | Ultimate Strength Design used. Building Classifications (4.4.1) are:  
• elastically responding  
• limited ductile, and  
• ductile.  
Strength reduction factor  
The strength reduction factor for flexure in beams and flexure and axial load in columns was 0.85. (The option of using a nominally unconfined column with a strength reduction factor of 0.7 was removed – clause 3.4.2.2.) The maximum ductility was set as 6 for concrete structures. This overrode the larger values permitted by NZS 4203:1992.  
Member stiffness  
Recommended section stiffness for seismic analysis was 0.4 times the gross section stiffness for rectangular beams and 0.35 for T and L beams. For columns the value varied from 0.4I g for an axial tension of ratio \( N^*/(A_g f_{c'}^d) \) of -0.05, 0.6I g at a ratio of 0.8, with interpolation for intermediate axial load ratios (clause C3.4.3.3).  
Bay elongation effects (i.e. elongation of plastic hinges in the beams pushing the columns apart).  
Requirements for the minimum length of support ledges for precast floor components to minimise the possibility of units supported on small ledges and/or on cover concrete (clause 4.3.6.4).  
Effective width of slab to contribute to beam moment flexural strength was increased and assumed to be the same in both loading directions (clause 8.5.3.3).
Effective anchorage of slab reinforcement required (clause 4.3.6.6).

Considerations were made for increase in shear force in the first storey columns and the formation of a plastic hinge forming in the columns adjacent to the first level beams (although these are not likely to govern) (Fenwick and MacRae, 2009).

**Details**

Confinement of columns increased for columns with a high axial load (refer to Section 7.5)

Confinement for gravity columns, which were not designed to resist seismic actions, was required (clause 8.4.7). Here, among other requirements, the spacing of transverse steel is no greater than (i) one third the minimum column cross sectional dimension, (ii) 10 times the longitudinal bar diameter.

Beam-column joint reinforcement requirements revised and reduced compared with the 1982 edition (clause 11.3.7)

Minimum seating lengths for precast floor components after reasonable allowance for construction tolerances were set as the larger of 1/180 of the clear span or 50 mm for solid slabs or hollow-core units and 75 mm for ribbed members (clause 4.3.6.4)

Stairs consider the seating lengths of NZS 4203:1992 (clause 4.4.13.2)

<table>
<thead>
<tr>
<th>Period</th>
<th>Loading Standard</th>
<th>Concrete Standard</th>
<th>Major changes</th>
</tr>
</thead>
</table>

For consistency with NZS 1170.5:2004 three classifications were defined for buildings. These relate to the value of the structural ductility factor used to determine the seismic design actions. They are:

- nominally ductile, using a design ductility of 1.25,
- limited ductile, and
- ductile buildings.

Three classifications of potential plastic regions were defined. Each of these have different detailing requirements and inelastic capacities (clause 2.6.1.3).

They are:

- nominally ductile plastic regions
- limited ductile plastic regions, and
- ductile plastic regions.

There is no direct connection between the type of plastic region and classification of a building:

- Design of brittle elements is excluded from this standard.
- Values for structural ductility factor of less than 1.25 are not given.
- $S_p$ values given in NZS 1170.5:2004 were replaced by 0.9 for a structural ductility factor, $\mu$, of 1.25, and 0.7 for a structural ductility factor of 3 or more, with linear interpolation between these limits (clause 2.6.2.2).
Welded wire fabric, with a strain capacity less than 10%, is permitted only in situations where it will not yield in ULS shaking or when, if it does yield or rupture, the integrity of the structure is not affected (clause 5.3.2.7).

**Member stiffness**

Minor revisions were made to the section stiffness where a high grade reinforcement was used (clause C6.9.1).

**Capacity design** (clause 2.6.5)

Contribution of prestressed floor components to overstrength of beams is considered (clause 9.4.1.6.2). The difference in effective widths of floor slabs contributing to nominal negative moment flexural strength of beams and to over-strength of beams is considered (clauses 9.4.1.6.1 and 9.4.1.6.2). Two methods are permitted for assessing capacity design actions in columns:

- The first method is based on the one contained in NZS 3101:1995 Appendix A with modifications to consider bi-axial actions more directly and to allow for the effects of elongation of beams on plastic hinge locations. In this method, each column above the primary plastic hinge located at its base of the column is proportioned and detailed with the aim of minimising inelastic deformation that may occur (Method A in Appendix D, clause D3.2 in the NZS 3101:2006).

- The second method permits a limited number of potential plastic hinges in the columns provided the remaining columns have sufficient nominal strength to ensure that the storey column sway shear strength exceeds the storey beam sway shear strength in each storey by a nominated margin. The beam-sway storey shear strength is calculated assuming over-strength actions are sustained in all the potential plastic regions associated with the storey being considered (refer to Appendix D, clause D3.3 in the NZS 3101:2006). This method has more restrictions on the lap positions of longitudinal bars and requiring more confinement reinforcement than the first method.

The significance of elongation of plastic hinges in beams on the actions in columns is recognised. In particular, elongation can cause plastic hinges, which are not identified in standard analyses, to form in columns immediately above or below the first elevated level. This can increase the shear forces induced in the columns. However, as the requirement for confinement reinforcement is generally more critical than shear reinforcement this is unlikely to be critical for the shear strength of these columns (refer to 10.4.7.1.2, B8.4, C2.6.1.3.3, C5.3.2, C10.4.6.6, C10.4.7.2.1 in the NZS 3101:2006).

In calculating overstrength actions in beams, allowance needs to be made for the possible material strengths and the increase in stress that may be sustained due to strain hardening. Strain levels are much higher in over-strength conditions than in normal ultimate strength design conditions. As strain levels increase the width of floor
Period  Loading Standard  Concrete Standard  Major changes

slab that acts with a beam increases. Consequently a greater width of slab needs to be assumed to contribute to overstrength than to design strength. This effect is recognised in the NZS 3101:2006 (clauses 9.4.1.6.1 and 9.4.1.6.2) but it was not recognised in earlier standards.

Precast prestressed floor units in a floor slab, which span past potential plastic hinges in a beam, can make a very significant difference to the over-strength capacity of plastic hinges. A method of assessing the strength due to this source is given in the Standard (clause 9.4.1.6.2).

Strength design
Primary plastic hinges detailed in terms of likely ultimate limit state inelastic demands. These demands are written in terms section curvature for a specified plastic hinge length, which is similar to specifying a plastic rotation (refer to clause 2.6.1).

Serviceability limit state with earthquake
New requirements for fully ductile (but not nominal or limited ductile structures) (clause 2.6.3.1).

The structural ductility that can be used in the ultimate limit state (ULS) is limited to 6 for buildings of normal importance; and in some cases a lower value is required (clause 2.6.1.2d).

For the serviceability limit state a structural ductility factor of 1 is required for SLS1, but a value of 2 may be used for SLS2 (clause 2.6.2.3.1). However, SLS2 is only applied to buildings of high importance (NZS 1170:2004, clause 5.2.1.4).

Clause 2.6.3.1 requires either that:

• the serviceability design strength is equal to, or exceeds, the serviceability design actions, or

• analysis shows that crack widths and deflections remaining after a serviceability limit state earthquake are acceptable considering the effect of inelastic deformation caused by moment redistribution and other shake down effects associated with repeated inelastic displacements during an earthquake.

Strength requirements for the serviceability limit state are related to the average strength of structural sections. This is taken as the nominal strength with a strength reduction factor of 1.1 (clause 2.6.3.2) to correspond to average material strengths.

Diaphragm Design
Similar material to NZS 3101:1995
Strut and tie analysis required for forces induced in the diaphragms associated with the ultimate limit-state, or with actions associated with overstrength in potential plastic regions (clause 13.3.3)

Floors containing precast prestressed units have special requirements (NZS 3101: 2006 plus Amendment 2) relating to (Fenwick and MacRae, 2009):

• limiting the possibility of the floors falling off supports (clause 18.7.4)

• limiting the possibility of brittle failure by:
  – requiring for low friction bearing strips with hollow-core units (clause 18.7.4)
<table>
<thead>
<tr>
<th>Period</th>
<th>Loading Standard</th>
<th>Concrete Standard</th>
<th>Major changes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>– requiring a thin linking slab between a precast unit and a parallel structural element, such as a beam or wall, which may deflect in a vertical direction relative to the precast unit. This is required to prevent the load transfer between the structural elements causing the precast units to fail (clause 18.6.7.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>– specifying requirements for shear strength of precast units in zones where over-strength actions can cause tensile stresses to be induced on the top surface of the precast units. In this situation the shear strength is reduced to a value comparable with a non-prestressed beam of the same dimensions (clause 19.3.11.2.4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>– specifying the position where reinforcement connecting the precast unit to the supporting structure is cut off or reduced is based on the capacity of the floor to sustain the negative moments and axial tension. These may be induced in the floor when over-strength actions act at the supports and vertical ground motion induces negative moments in the floor (clause 19.4.3.6)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>– cautioned against supporting precast units on structural elements that may deform and induce torsional moments as these may lead to torsional failure of the floor unit. This situation can be critical for hollow-core flooring (clause C19.4.3.6).</td>
</tr>
</tbody>
</table>
Table C5A.2: Comparison of concrete property requirements and design specifications from four generations of New Zealand standards post-1970

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Specified compressive strength (MPa)</td>
<td>$f'_c = 17.2$ MPa, 20.7 MPa, 27.6 MPa, 34.5 MPa</td>
<td>$20$ MPa &lt; $f'_c &lt; 55$ MPa</td>
<td>$17.5$ MPa &lt; $f'_c &lt; 100$ MPa</td>
<td>$25$ MPa ≤ $f'_c &lt; 100$ MPa</td>
</tr>
</tbody>
</table>
| Modulus of rupture (MPa) | For normal weight concrete: $f_r = 0.8 \sqrt{f'_c}$  
  For lightweight concrete:  
  - where $f_{ct}$ is specified and the concrete mix designed in accordance with NZS 3152: $f_r = 0.8 \times 1.8 f_{ct}$ (the value of $1.8 f_{ct}$ shall not exceed $\sqrt{f'_c}$)  
  - where $f_{ct}$ is not specified, $f_r$ shall be multiplied by $0.75$ (for all-lightweight concrete) $0.85$ (for sand-lightweight concrete)  
  • $f_r = 0.6 \lambda \sqrt{f'_c}$ (for the purpose of calculation deflections)  
  • $\lambda = 0.85$ (normal weight sand, lightweight coarse aggregate)  
  • $\lambda = 0.75$ (lightweight sand, lightweight coarse aggregate)  
  • $\lambda = 1.0$ (concrete with no lightweight aggregates)  
  • $f_r = 1.12 f_{ct}$ (when the indirect tensile strength of concrete, $f_{ct}$, specified and lightweight concrete is used, but no more than $0.6 \lambda \sqrt{f'_c}$)  
  • from testing  
    - modulus of rupture test (AS 1012: Part 11); or  
    - indirect tensile strength test (AS 1012: Part 10) |
| Direct tensile strength (MPa) | $E = 0.043 w^{1.5} \sqrt{f'_c}$ (for 1450 < w < 2500 kg/m$^3$)  
  $E = 0.043 w^{1.5} \sqrt{f'_c}$ (for 1400 < w < 2500 kg/m$^3$)  
  $E = (3320 \sqrt{f'_c} + 6900) (\rho^{1.5}$ (for 1400 < $\rho < 2500$ kg/m$^3$)  
  $E = (3320 \sqrt{f'_c} + 6900)$ (for normal weight concrete)  
  $E \geq$ value corresponding to $(f'_c + 10)$ MPa (when strain induced action are critical)  
  Note: For the serviceability limit state, this value may be used in lieu of above expression. |
<p>| Elastic Modulus | $E = \frac{0.36 \sqrt{f'_c}}{\rho}$ or $(0.54 \times \sqrt{f'_c})$ (indirect tensile strength obtained from Brazil test according to AS 1012: Part 10) | $E = \frac{0.36 \sqrt{f'_c}}{\rho}$ or $(0.54 \times \sqrt{f'_c})$ (indirect tensile strength obtained from Brazil test according to AS 1012: Part 10) | $E = \frac{0.36 \sqrt{f'_c}}{\rho}$ or $(0.54 \times \sqrt{f'_c})$ (indirect tensile strength obtained from Brazil test according to AS 1012: Part 10) | $E = \frac{0.36 \sqrt{f'_c}}{\rho}$ or $(0.54 \times \sqrt{f'_c})$ (indirect tensile strength obtained from Brazil test according to AS 1012: Part 10) |</p>
<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poisson ratio</td>
<td>0.2</td>
<td>0.2 (for normal density concrete) Shall be determined (for lightweight concrete)</td>
</tr>
</tbody>
</table>
| Coefficient of thermal expansion \((1/°C)\) | \(12 \times 10^{-6}\)     | For concrete of an aggregate type:  
- Greywacke \((9.5 - 11 \times 10^{-6})\)  
- Phonolite \((10.0 - 11.0 \times 10^{-6})\)  
- Basalt \((9.0 - 10.0 \times 10^{-6})\)  
- Andesite \((7.0 - 9.0 \times 10^{-6})\)  
The coefficient of thermal expansion may be taken as \(12 \times 10^{-6}/°C\) or determined from suitable test data for other aggregate types. For self-compacting concrete these values shall be increased by 15%. |
| Shrinkage                      |                            | The design unrestrained shrinkage strain may be determined by testing to AS 1012 Part 13, or appropriate published values.                                                                                   |
| Creep                          |                            | The creep coefficient used for design may be determined by testing to AS 1012 Part 16, or to ASTM C512, or assessed from appropriate published values.                                                        |
| Stress-strain curves           |                            | • Assumed to be of curvilinear form defined by recognised simplified equations; or  
• Determined from suitable test data.                                                                                                |
| Applicable density range \((kg/m^3)\) | 1800 to 2800               |                                                                                                                                                                                                              |

**Note:**  
1. Formulas have been converted to metric units.  
2. \(w\): weight of concrete.
## Appendix C5B: Historical Requirements for Concrete Strength Testing in New Zealand

### Table C5B.1: Concrete strength tests for proof of control

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of test specimens</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 specimens made from one sample of concrete</td>
<td>Same as NZS 3104:1983</td>
<td>Same as NZS 3104:1983</td>
<td></td>
</tr>
<tr>
<td>2 specimens when the number of tests &gt; 20 and the 28-day compressive testing mean has a within-test coefficient of variation of the test series of less than 4%.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frequency of testing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Ready-mixed concrete:</td>
<td>Same as NZS 3104:1983</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/75 m³ (up to 15,000 m³ per annum), with an additional test for every 250 m³ above 15,000 m³</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>At least 120 tests per annum</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Site-mixed concrete:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 sample (each day/75 m³)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Ready-mixed concrete:</td>
<td>Same as NZS 3104:1983</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Same as NZS 3104:1983</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>At least 10 tests per month (6 tests per month in the case of plants producing less than 9000 m³ per annum)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Site-mixed concrete:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Same as NZS 3104:1983</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Appendix C5C: Material Test Methods

### C5C.1 Concrete

Table C5C.1: Overview of destructive, semi-destructive and non-destructive tests for investigating concrete material properties (De Pra, Bianchi and Pampanin, 2015; Malek et al., 2015)

<table>
<thead>
<tr>
<th>Method</th>
<th>Capability/Use</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DESTRUCTIVE TESTS</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive test</td>
<td>Strength of concrete</td>
<td>Direct evaluation of concrete strength from compressive tests on cylindrical specimens</td>
<td>Disturbance of the sample, so excessive damage to obtain a representative core of concrete. Previous test with pacometer necessary to individuate the regions without bars.</td>
</tr>
<tr>
<td><strong>SEMI-DESTRUCTIVE TESTS</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pull-out</td>
<td>In-place estimation of the compressive and tensile strengths</td>
<td>In-place strength of concrete can be quickly measured</td>
<td>Pull-out device must be inserted in a hole drilled in the hardened concrete. Only a limited depth of material can be tested.</td>
</tr>
<tr>
<td>Pull-off/tear-off</td>
<td>Direct tension test</td>
<td>In situ tensile strength of concrete, Determining bond strength between existing concrete and repair material</td>
<td>Sensitivity to rate of loading</td>
</tr>
<tr>
<td>Penetration probe (Windsor probe)</td>
<td>Estimation of compressive strength, uniformity and quality of concrete, Measuring the relative rate of strength development of concrete at early ages</td>
<td>The equipment is easy to use (not requiring surface preparation), The results are not subject to surface conditions and moisture content</td>
<td>Minimum edge distance and member thickness are requested, Not precise prediction of strength for concrete older than 5 years and where surface is affected by carbonation or cracking</td>
</tr>
<tr>
<td><strong>NON-DESTRUCTIVE TESTS</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Visual tests</td>
<td>The first step in investigating concrete material</td>
<td>Quick evaluation of damage</td>
<td>No detailed information</td>
</tr>
<tr>
<td>Rebound hammer</td>
<td>Measuring surface hardness of concrete to estimate compressive strength</td>
<td>The assessment of the surface layer strength</td>
<td>Results can only suggest the hardness of surface layer</td>
</tr>
<tr>
<td>Durability test</td>
<td>Concrete electrical resistivity</td>
<td>Measuring the ability of the concrete to conduct the corrosion current</td>
<td>Inexpensive, simple and many measurements can be made rapidly.</td>
</tr>
<tr>
<td>Permeability</td>
<td>To evaluate the transfer properties of concrete (porosity)</td>
<td>Useful method to evaluate the risk of leaching, corrosion and freezing</td>
<td>Thickness limitation, Age, temperature dependent, Sufficient lateral sealing</td>
</tr>
<tr>
<td>Method</td>
<td>Capability/Use</td>
<td>Advantages</td>
<td>Disadvantages</td>
</tr>
<tr>
<td>----------------------------</td>
<td>--------------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------</td>
<td>--------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Fiberscope</td>
<td>To check the condition of cavities, and honeycombing in reinforced concrete</td>
<td>Direct visual inspection of inaccessible parts of an element</td>
<td>Semi destructive as the probe holes usually must be drilled</td>
</tr>
<tr>
<td></td>
<td>Voids detection along grouted post-stressed tendons</td>
<td></td>
<td>Needs additional fibre to carry light from an external source inspected</td>
</tr>
<tr>
<td>Spectral analysis of surface waves</td>
<td>Evaluation of concrete strength and quality Identification of internal damage and location of reinforcement</td>
<td>Excellent for determining the quality and uniformity of concrete; especially for rapid survey of large areas and thick members</td>
<td>The measure can be distorted by the presence of lesions in the concrete</td>
</tr>
<tr>
<td></td>
<td>Quality control and integrity of concrete</td>
<td>Access to only one face is needed</td>
<td>The test requires smooth surfaces for a good adhesion of the probes</td>
</tr>
<tr>
<td></td>
<td>Access to only one face is needed Internal discontinuities and their sizes can be estimated</td>
<td></td>
<td>No information about the depth of suspected flaw</td>
</tr>
<tr>
<td>Impact echo method</td>
<td>Defects within concrete element such as delamination, voids, honeycombing</td>
<td>Access to only one face is needed</td>
<td>The ability of instrument is limited to less than 2 m thickness</td>
</tr>
<tr>
<td>Spectral analysis of surface waves</td>
<td>Determining the stiffness profile of a pavement</td>
<td>Capability of determining the elastic properties of layered systems such as pavement and interlayered concrete</td>
<td>Complex signal processing</td>
</tr>
<tr>
<td></td>
<td>Depth of deteriorated concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gamma radiography</td>
<td>Location of internal cracks, voids and variations in density of concrete</td>
<td>Simple to operate</td>
<td>X-ray equipment is bulky and expensive</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Applicable to a variety of materials</td>
<td>Difficult to identify cracks perpendicular to radiation beam</td>
</tr>
<tr>
<td>Backscatter radiometry</td>
<td>Determining in-place density of fresh or hardened concrete</td>
<td>Access only to surface of test object</td>
<td>The accuracy of this method is lower than direct transmission</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Since this method’s measurements are affected by the top 40 to 100 mm, best for assessing surface zone of concrete element</td>
<td>Measurements are influenced by near surface material and are sensitive to chemical composition</td>
</tr>
<tr>
<td>CT scanning</td>
<td>Concrete imaging</td>
<td>3D crack/damage monitoring</td>
<td>Sophisticated software for analysis</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Not in situ application</td>
<td>Not in situ application</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Access to CT scanner needed</td>
<td>Access to CT scanner needed</td>
</tr>
<tr>
<td>Infrared thermography</td>
<td>Detecting delamination, heat loss and moisture movement through concrete elements; especially for flat surfaces</td>
<td>Permanent records can be made</td>
<td>Expensive technique</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tests can be done without direct access to surface by means of infrared cameras</td>
<td>Reference standards are needed</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Very sensitive to thermal interference from other heat sources</td>
</tr>
<tr>
<td>Method</td>
<td>Capability/Use</td>
<td>Advantages</td>
<td>Disadvantages</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>-------------------------------------------------------------------------------</td>
<td>-----------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Ground penetrating radar</td>
<td>Identification of location of reinforcement, depth of cover, location of voids and cracks, Determination of in situ density and moisture content</td>
<td>Can survey large areas rapidly</td>
<td>Results must be correlated to test results on samples obtained</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Low level signals from targets as depth increases</td>
</tr>
<tr>
<td>Acoustic emission</td>
<td>Real time monitoring of concrete degradation growth and structural performance</td>
<td>A few transducers are enough to locate defects over large areas, Detecting the initiation and growth of cracks in concrete under stress</td>
<td>Passive technique, could be used when the structure is under loading</td>
</tr>
<tr>
<td>Ultrasonic tomography (MIRA)</td>
<td>Uses high frequency (greater than 20,000 Hz) sound waves to characterise the properties of materials or detect their defects</td>
<td>Thickness measurement, reinforcement location, and distress evaluation</td>
<td>Significant efforts and user expertise are required for measurement and data interpretation of large scale application</td>
</tr>
<tr>
<td>Petrography</td>
<td>Forensic investigation of concrete, Determining the composition and identifying the source of the materials, Determination of w/c, Determining the depth of fire damage</td>
<td>Microscopic examination of concrete samples</td>
<td>Laboratory facilities as well as highly experienced personnel are needed to interpret the result</td>
</tr>
<tr>
<td>Sclerometric method</td>
<td>Determination of compressive strength</td>
<td>Determination of a sclerometric index connected to compressive strength</td>
<td>The instrument must be in the horizontal direction or the reliability of results is reduced</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Empirical formulas, based on probabilistic methods, are used to obtain the concrete strength</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>The preparation of the test surface is laborious and expensive</td>
</tr>
<tr>
<td>SonReb method</td>
<td>Determination of compressive strength</td>
<td>The concomitant use of sclerometric and ultrasonic methods can reduce mistakes due to the influence of humidity and aging of concrete</td>
<td>Risk of regression on a small statistically representative sample</td>
</tr>
</tbody>
</table>
C5C.2 Reinforcing Steel

Table C5C.2: Destructive and non-destructive tests for investigating reinforcing steel material properties (De Pra, Bianchi and Pampanin, 2015)

<table>
<thead>
<tr>
<th>Method</th>
<th>Capability/Use</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DESTRUCTIVE TESTS</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile test</td>
<td>Steel strength (yield strength, tensile strength and elongation on 5 diameters gauge length)</td>
<td>Direct evaluation of steel strength</td>
<td>The test is limited to areas that are easily accessible</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>The interpretation of the results is subjective and depends on the operator’s experience</td>
</tr>
<tr>
<td><strong>NON-DESTRUCTIVE TESTS</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hardness stress with Leeb method</td>
<td>Evaluation of hardness and tensile strength</td>
<td>Low cost</td>
<td>A previous survey with pacometer is required to identify the regions with less cover</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>The device is portable, so particularly useful in difficult operative conditions</td>
</tr>
<tr>
<td>Penetrating liquids</td>
<td>Deterioration of steel</td>
<td>Simple to apply</td>
<td>The surface must be cleaned before the test to remove all extraneous substances</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Not applicable on too porous surfaces</td>
</tr>
<tr>
<td>Measure of potential corrosion of reinforcement</td>
<td>Evaluation of potential corrosion</td>
<td>Possibility to measure the potential corrosion of the bars</td>
<td>The electrode must be dampened 12 hours before the test</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A previous survey with a pacometer is required to individuate the presence of bars</td>
</tr>
<tr>
<td>Survey with pacometer</td>
<td>Identification of bars (cover, bar free interface, spacing of stirrups, diameters of bars)</td>
<td>Identification of the areas without bars in order to identify where it is possible to carry out concrete tests</td>
<td>The device is sensitive to the presence of the ferromagnetic material</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>The method is slow and laborious</td>
</tr>
<tr>
<td>Georadar</td>
<td>Determination of dimensions and depth of foundations</td>
<td>Possible to have information on foundations</td>
<td>Calibration of the instrumentation is required before the data acquisition, investigating two directions</td>
</tr>
</tbody>
</table>
**Appendix C5D: History of New Zealand Reinforcement Requirements**

### C5D.1 Mechanical Properties of Steel Reinforcing Bars

#### Table C5D.1: Mechanical properties of steel reinforcement bars – pre-1960s

<table>
<thead>
<tr>
<th>Steel Property</th>
<th>Standard Steel</th>
<th>NZS 197:1949 (BS 785:1938)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of steel</td>
<td>Plain round bar</td>
<td>Mild steel (MS) Medium tensile (MT) High tensile (HT)</td>
</tr>
<tr>
<td>Yielding stress</td>
<td>Bar size (diameter)</td>
<td>MS</td>
</tr>
<tr>
<td></td>
<td>Up to 1 inch</td>
<td>Not Specified</td>
</tr>
<tr>
<td></td>
<td>Over 1 to 1½ inch</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Over 1½ to 2 inch</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Over 2 to 2½ inch</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Over 2½ to 3 inch</td>
<td></td>
</tr>
<tr>
<td>Tensile strength</td>
<td>≥ 28 tsi (≈ 386 MPa)</td>
<td>≥ 33 tsi (≈ 455 MPa)</td>
</tr>
<tr>
<td></td>
<td>≤ 33 tsi (≈ 455 MPa)</td>
<td>≤ 38 tsi (≈ 524 MPa)</td>
</tr>
<tr>
<td>Elongation at fracture (%)</td>
<td>Up to 1 inch</td>
<td>≥ 20(1)</td>
</tr>
<tr>
<td></td>
<td>Over 1 to 1½ inch</td>
<td>≥ 16(1)</td>
</tr>
<tr>
<td></td>
<td>Under ¾ inch</td>
<td>≥ 24(2)</td>
</tr>
</tbody>
</table>

**Note:**
- psi = pounds per square inch
- tsi = tons per square inch
- 1. Measured on a minimum 8 diameters gauge length.
- 2. Measured on a minimum 4 diameters gauge length.
Table C5D.2: Mechanical properties of steel reinforcement bars – 1960s to mid-1970s

<table>
<thead>
<tr>
<th>Standard Property</th>
<th>NZS 197:1949 (BS 785:1938)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of steel</td>
<td>Plain round bar</td>
<td>Mild steel (MS)</td>
<td>Medium tensile (MT)</td>
</tr>
<tr>
<td>Yielding stress</td>
<td>Bar size (diameter)</td>
<td>MS</td>
<td>MT</td>
</tr>
<tr>
<td></td>
<td>Up to 1 inch</td>
<td>Not Specified</td>
<td>19.5 tsi (=270 MPa)</td>
</tr>
<tr>
<td></td>
<td>Over 1 to 1½ inch</td>
<td></td>
<td>18.5 tsi (=255 MPa)</td>
</tr>
<tr>
<td></td>
<td>Over 1½ to 2 inch</td>
<td></td>
<td>17.5 tsi (=241 MPa)</td>
</tr>
<tr>
<td></td>
<td>Over 2 to 2½ inch</td>
<td></td>
<td>16.5 tsi (=227 MPa)</td>
</tr>
<tr>
<td></td>
<td>Over 2½ to 3 inch</td>
<td></td>
<td>16.5 tsi (=227 MPa)</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>≥ 28 tsi (≈ 386 MPa)</td>
<td>≥ 33 tsi (≈ 455 MPa)</td>
<td>≥ 37 tsi (≈ 510 MPa)</td>
</tr>
<tr>
<td></td>
<td>≤ 33 tsi (≈ 455 MPa)</td>
<td>≤ 38 tsi (≈ 524 MPa)</td>
<td>≤ 43 tsi (≈ 593 MPa)</td>
</tr>
<tr>
<td>Elongation at fracture (%)</td>
<td>Up to 1 inch</td>
<td>≥ 20(1)</td>
<td>≥ 18(1)</td>
</tr>
<tr>
<td></td>
<td>Over 1 to 1½ inch</td>
<td>≥ 16(1)</td>
<td>≥ 14(1)</td>
</tr>
<tr>
<td></td>
<td>Under ¾ inch</td>
<td>≥ 24(2)</td>
<td>≥ 22(2)</td>
</tr>
</tbody>
</table>

Note:
- psi = pounds per square inch
- tsi = tons per square inch
- Measured on a minimum 8 diameters gauge length.
- Measured on a minimum 4 diameters gauge length.

1. Measured on a minimum 8 diameters gauge length.
2. Measured on a minimum 4 diameters gauge length.
### Table C5D.3: Mechanical properties of steel reinforcement bars –1970s onwards

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of steel</td>
<td>Grade 275</td>
<td>Grade 380</td>
<td>Grade 300</td>
</tr>
<tr>
<td></td>
<td>Grade 380</td>
<td></td>
<td>Grade 430</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Grade 300</td>
<td>Grade 300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Grade 500</td>
<td>Grade 500</td>
</tr>
<tr>
<td>Yielding stress (MPa)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower bound</td>
<td>≥ 275(^{(\text{min})}) (300(^{(k)}))</td>
<td>≥ 410(^{(\text{min})}) (430(^{(k)}))</td>
<td>≥ 300(^{(l)})</td>
</tr>
<tr>
<td>Upper bound</td>
<td>≤ 380(^{(\text{max})}) (355(^{(l)}))</td>
<td>≤ 520(^{(\text{max})}) (500(^{(k)}))</td>
<td>≤ 380(^{(k)})</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>≥ 380 (\leq 520)</td>
<td>≥ 570*</td>
<td>Not specified</td>
</tr>
<tr>
<td>Ratio (R_m/R_e) (TS/YS)</td>
<td>Not specified</td>
<td>1.15 ≤ (\frac{TS}{YS}) ≤ 1.50</td>
<td>1.15 ≤ (\frac{TS}{YS}) ≤ 1.40</td>
</tr>
<tr>
<td>Elongation at maximum force (A_{gt}) (%)</td>
<td>Not specified</td>
<td>Not specified</td>
<td>≥ 15</td>
</tr>
<tr>
<td>Elongation at fracture (%)</td>
<td>≥ 20(^{(1)})</td>
<td>≥ 12(^{(1)})</td>
<td>≥ 20(^{(1)})</td>
</tr>
</tbody>
</table>

**Note:**

* But not less than 1.2 times the actual yield stress
1. Measured on a minimum 4 diameters gauge length.

\(k\) characteristic value

\(TS\) = tensile strength

\(YS\) = yield stress

\(R_m\) = value of maximum tensile strength *(determined from a single tensile test in accordance with AS 1391)*

\(R_e\) = value of the yield stress or 0.2% proof stress *(determined from a single tensile test in accordance with AS 1391)*
### C5D.2 Beam Reinforcement and Detailing

**Table C5D.4: Evolution of standard-based details requirements for beams**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral support spacing</td>
<td>$50b_w$</td>
<td>$50b_w$ (for earthquake)</td>
<td>$50b_w$</td>
<td></td>
</tr>
<tr>
<td>$\rho_{\text{max}}$</td>
<td>$\rho_{\text{max}} = 0.75 \rho_{\text{bal}}$</td>
<td>$\rho_{\text{max}} = 0.75 \rho_{\text{bal}}$</td>
<td>$\rho_{\text{max}} = 0.75 \rho_{\text{bal}}$</td>
<td>$\rho_{\text{max}} = 0.75 \rho_{\text{bal}}$ (for USD)</td>
</tr>
<tr>
<td>$\rho_{\text{min}}$</td>
<td>$\rho_{\text{min}} = \frac{\sqrt{f_{\text{c}}}}{f_y} \geq \frac{1.4}{f_y}$</td>
<td>$\rho_{\text{min}} = \frac{\sqrt{f_{\text{c}}}}{f_y} \geq \frac{1.4}{f_y}$</td>
<td>$\rho_{\text{min}} = \frac{1.4}{f_y}$</td>
<td>$\rho_{\text{min}} = \frac{200}{f_y}$</td>
</tr>
<tr>
<td>$\rho_{\text{min}}$ (alternatively)</td>
<td>$\rho_{\text{min}} = \frac{4}{3} \rho_{\text{reqd}}$ (for gravity only)</td>
<td>$\rho_{\text{min}} = \frac{4}{3} \rho_{\text{reqd}}$ (for gravity only)</td>
<td>$\rho_{\text{min}} = \frac{4}{3} \rho_{\text{reqd}}$</td>
<td>$\rho_{\text{min}} = \frac{4}{3} \rho_{\text{reqd}}$</td>
</tr>
<tr>
<td>Maximum $d_b$ in internal beam-column joints (for nominally ductile structures)</td>
<td>$\frac{d_b}{h_c} = 4 \alpha_f \frac{\sqrt{f_{\text{c}}}}{f_y}$</td>
<td>$\frac{d_b}{h_c} = 4 \alpha_f \frac{\sqrt{f_{\text{c}}}}{f_y}$</td>
<td>$\frac{d_b}{h_c} = 4 \alpha_f \frac{\sqrt{f_{\text{c}}}}{f_y}$</td>
<td>$\frac{d_b}{h_c} = 4 \alpha_f \frac{\sqrt{f_{\text{c}}}}{f_y}$</td>
</tr>
<tr>
<td></td>
<td>$\alpha_f = 0.85$ (two-way)</td>
<td>$\alpha_f = 0.85$ (two-way)</td>
<td>$\alpha_f = 1.00$ (one-way)</td>
<td>$\alpha_f = 1.00$ (one-way)</td>
</tr>
<tr>
<td>Minimum requirements for transverse reinforcement</td>
<td>5mm in diameter</td>
<td>6mm in diameter (for earthquake)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum nominal shear stress</td>
<td>$v_n \leq 0.2 f'_{\text{c}}$ or 8MPa</td>
<td>$v_n \leq \left{ \begin{array}{l} 0.2 f'<em>{\text{c}} \ 1.14 \frac{f'</em>{\text{c}}}{f_y} \ 9M Pa \end{array} \right.$</td>
<td>$v_n \leq 0.2 f'_{\text{c}}$ or 6MPa</td>
<td>$v_n \leq 5 \sqrt{f_{\text{c}}}$ or $v_n \leq 8.5 \sqrt{f_{\text{c}}}$ (USD)</td>
</tr>
<tr>
<td>Spacing limits for shear reinforcement</td>
<td>$S_{\text{max}} \leq \left{ \begin{array}{l} 0.5d, b_w \ 500\text{ mm} \ 16d \end{array} \right.$</td>
<td>$S_{\text{max}} \leq \left{ \begin{array}{l} 0.5d \ 600\text{ mm} \end{array} \right.$</td>
<td>$S_{\text{max}} \leq \left{ \begin{array}{l} 0.5d \ (600\text{ mm}) \end{array} \right.$</td>
<td>$S_{\text{max}} \leq \left{ \begin{array}{l} 0.75d \ (450\text{ mm}) \end{array} \right.$</td>
</tr>
<tr>
<td>(0.5d and 500 mm reduced by half if $v_s \geq 0.33 \sqrt{f_{\text{c}}}$)</td>
<td>(0.5d and 500 mm reduced by half if $v_s \geq 0.07 f'_{\text{c}}$)</td>
<td>(0.5d and 500 mm reduced by half if $v_s \geq 0.07 f'_{\text{c}}$)</td>
<td></td>
<td>(S$<em>{\text{max}} \leq 0.25d$ if $v_n \geq 3 \sqrt{f</em>{\text{c}}}$ or $v_n \geq 5.1 \sqrt{f_{\text{c}}}$ for USD)</td>
</tr>
<tr>
<td>Minimum area of shear reinforcement</td>
<td>$A_v = \frac{1}{16} \sqrt{f_{\text{c}}} \frac{b_{\text{ws}}}{f_{\text{yt}}}$</td>
<td>$A_v = 0.35 \frac{b_{\text{ws}}}{f_{\text{yt}}}$</td>
<td>$A_v = 0.35 \frac{b_{\text{ws}}}{f_{\text{yt}}}$</td>
<td>$A_v = 0.0015b_{\text{ws}}$</td>
</tr>
<tr>
<td>-------------------------------------------------</td>
<td>---------------</td>
<td>---------------</td>
<td>---------------</td>
<td>---------------</td>
</tr>
<tr>
<td>Dimension of beams (for earthquake)</td>
<td>$\frac{L_n}{b_w} \leq 25$</td>
<td>$\frac{L_n}{b_w} \leq 25$</td>
<td>$\frac{L_n}{b_w} \leq 25$</td>
<td>$\frac{L_n}{b_w} \leq 25$</td>
</tr>
<tr>
<td>$\frac{L_{nh}}{b_{w'}} \leq 100$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$b_{w} \geq 200$ mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\rho_{\text{max}}$ (for earthquake, within plastic hinge region)</td>
<td>$\rho_{\text{max}} = \frac{f'_c + 10}{6f_y} \leq 0.025$</td>
<td>$\rho_{\text{max}} = \frac{f'_c + 10}{6f_y} \leq 0.025$</td>
<td>$\rho_{\text{max}} = \frac{1 + 0.17\left(\frac{f'_c}{f_y} - 3\right)}{\left(1 + \frac{\rho'}{\rho}\right)} \leq \frac{7}{f_y}$</td>
<td></td>
</tr>
<tr>
<td>$\rho_{\text{min}}$ (for earthquake, within plastic hinge region)</td>
<td>$A'_s &gt; 0.5A_s$ for ductile plastic regions. $A'_s &gt; 0.38A_s$ for limited ductile plastic regions.</td>
<td>$A'_s &gt; 0.5A_s$</td>
<td>$A'_s &gt; 0.5A_s$</td>
<td>$\rho_{\text{min}} = \frac{1.4}{f_y}$</td>
</tr>
<tr>
<td>$\rho_{\text{min}} = \sqrt{\frac{f'_c}{4f_y}}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum longitudinal beam bar diameter to column depth (for earthquake)</td>
<td>$\frac{d_b}{h_c} \leq 3.3\alpha_d\sqrt{\frac{f'_c}{1.25f_y}}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f'_c \leq 70$ MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\alpha_d = 1.00$ (ductile)</td>
<td></td>
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</tr>
<tr>
<td>$\alpha_d = 1.20$ (limited ductile)</td>
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<tr>
<td>Minimum area of shear reinforcement (for shear reinforcement)</td>
<td>$A_v = \frac{1}{12} \sqrt{\frac{f_c}{f_y}} \frac{b_w s}{t_{vc}}$</td>
<td></td>
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</tr>
<tr>
<td>Spacing limits for shear reinforcement (for shear reinforcement)</td>
<td>$S_{\text{max}} = \frac{12d_b}{d/2}$</td>
<td>$S_{\text{max}} = \frac{16d_b}{b_w}$</td>
<td>$S_{\text{max}} \leq \frac{48d_v}{16d_b}$</td>
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<tr>
<td>---------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Minimum area of shear reinforcement in plastic hinge regions (for earthquake)</td>
<td>( A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b} )</td>
<td>( A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b} )</td>
<td>( A_{te} = \frac{\sum A_b f_y}{160 f_{yt}} \frac{s}{d_b} )</td>
<td>( A_{te} = \frac{\sum A_b f_y}{160 f_{yt}} \frac{s}{d_b} )</td>
</tr>
<tr>
<td>Spacing limits for shear reinforcement in plastic hinge regions (for earthquake)</td>
<td>( S_{max} = \left{\begin{array}{l} 6d_b / d / 4 \end{array}\right} \text{ (ductile)} )</td>
<td>( S_{max} = \left{\begin{array}{l} 6d_b / d / 4 \end{array}\right} \text{ (limited ductile)} )</td>
<td>( S_{max} = \left{\begin{array}{l} 10d_b / d / 4 \end{array}\right} \text{ (limited ductile)} )</td>
<td>( S_{max} = \left{\begin{array}{l} 6d_b / d / 4 \end{array}\right} )</td>
</tr>
<tr>
<td>Maximum nominal shear stress (for earthquake)</td>
<td>( v_b \leq \left{\begin{array}{l} 0.16 f'_{c} \end{array}\right} )</td>
<td>( v_b \leq \left{\begin{array}{l} 0.16 f'_{c} \end{array}\right} )</td>
<td>( v_b \leq \left{\begin{array}{l} 0.85 \sqrt{f'_{c}} \end{array}\right} )</td>
<td>( v_b \leq \left{\begin{array}{l} 0.85 \sqrt{f'_{c}} \end{array}\right} )</td>
</tr>
</tbody>
</table>

**Note:**

NZS 3101P:1970 units of [psi]

USD: Ultimate Strength Design
### C5D.3 Column Reinforcement and Detailing

Table C5D.5: Evolution of standards-based details requirements for columns (Niroomandi et al., 2015)

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<tr>
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</thead>
<tbody>
<tr>
<td><strong>Strength reduction factor ( (\phi) )</strong></td>
<td>0.85</td>
<td>0.85</td>
<td>0.9 for conforming transverse 0.7 for others</td>
<td>0.75 for spirally reinforced 0.7 for tied</td>
<td>-</td>
</tr>
<tr>
<td><strong>( f'_c )</strong></td>
<td>25 – 100 MPa</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>For DPRs and LDPRs:</td>
<td>25 – 70 MPa</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>( f_y )</strong></td>
<td>&lt; 500 MPa</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>( f_y ) _</strong></td>
<td>&lt; 500 MPa for shear &lt; 800 MPa for confinement</td>
<td>&lt; 500 MPa for shear &lt; 800 MPa for confinement</td>
<td>&lt; 400 MPa</td>
<td>&lt; 414 MPa</td>
<td>-</td>
</tr>
<tr>
<td><strong>Maximum axial compressive load</strong></td>
<td>( 0.85\phi N_{n,max} ) _ for DPRs and LDPRs: ( 0.7\phi N_{n,max} ) _</td>
<td>( 0.85\phi N_{n,max} ) _ for DPRs: ( 0.7\phi N_{n,max} ) _</td>
<td>( 0.85\phi N_{n,max} ) _ for conforming, otherwise ( 0.8\phi N_{n,max} ) _</td>
<td>( P_0^{18} ) _ for tied columns: ( P = c A_c + n c A ) _ for spirally columns: ( P = c A_k = n c A + 2t_b A_b )</td>
<td></td>
</tr>
<tr>
<td><strong>Dimension of column</strong></td>
<td>( b_w \geq L_n/25 ) _ ( b_w \geq \sqrt{L_n h}/100 ) _</td>
<td>( b_w \geq L_n/25 ) _ ( b_w \geq \sqrt{L_n h}/100 ) _</td>
<td>( b_w \geq L_n/25 ) _ ( b_w \geq \sqrt{L_n h}/100 ) _</td>
<td>25.4 mm for circular 20.32 for rectangular or ( A_g &gt; 413 \text{ mm}^2 ) _</td>
<td>-</td>
</tr>
<tr>
<td><strong>Extend of ductile detailing length, ( l_y ), for detailing purposes</strong></td>
<td>( l_y = h ) for ( N_0^c \leq 0.25\phi f'_c A_g ) _ ( l_y = 2h ) for ( 0.25\phi f'_c A_g &lt; N_0^c \leq 0.5\phi f'_c A_g ) _ ( l_y = 3h ) for ( N_0^c &gt; 0.5\phi f'_c A_g ) _</td>
<td>( l_y = h ) for ( N_0^c \leq 0.25\phi f'_c A_g ) _ ( l_y = 2h ) for ( 0.25\phi f'_c A_g &lt; N_0^c \leq 0.5\phi f'_c A_g ) _ ( l_y = 3h ) for ( N_0^c &gt; 0.5\phi f'_c A_g ) _</td>
<td>( l_y = h ) for ( P_{c13} \leq 0.3\phi f'_c A_g ) _ ( l_y = 1.5h ) for ( P_c &gt; 0.3\phi f'_c A_g ) _</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td><strong>Minimum longitudinal reinforcement ratio</strong></td>
<td>0.008( A_g ) _</td>
<td>0.008( A_g ) _</td>
<td>0.008( A_g ) _</td>
<td>0.01 _</td>
<td>0.008 _</td>
</tr>
<tr>
<td>-------------------------------------------------</td>
<td>----------------</td>
<td>----------------</td>
<td>----------------</td>
<td>----------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>Maximum longitudinal reinforcement ratio</td>
<td>$0.08A_g$</td>
<td>$0.08A_g$</td>
<td>$0.08A_g$</td>
<td>0.08</td>
<td>0.08</td>
</tr>
<tr>
<td>For DPRs and LDPRs:</td>
<td>$18 A_g/f_y$</td>
<td>$18 A_g/f_y$</td>
<td>$18 A_g/f_y$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>For DPRs:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum longitudinal reinforcement ratio at splices</td>
<td>$0.08A_g$</td>
<td>$0.08A_g$</td>
<td>$0.08A_g$</td>
<td>0.12</td>
<td>-</td>
</tr>
<tr>
<td>For DPRs and LDPRs:</td>
<td>$24 A_g/f_y$</td>
<td>$24 A_g/f_y$</td>
<td>$24 A_g/f_y$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>For DPRs:</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Minimum number of longitudinal bars</td>
<td>8 bars, but may be reduced 6 or 4 if clear spacing is less than 150 mm and $N^* \leq 0.1f'_c A_g$</td>
<td>6 bars in a circular arrangement</td>
<td>6 bars in a circular arrangement</td>
<td>Same as nominally ductile (1995)</td>
<td>Same as nominally ductile (1995)</td>
</tr>
<tr>
<td>Maximum spacing between longitudinal bars requiring restraint</td>
<td>Circular columns, larger of one quarter of a diameter or 200 mm</td>
<td>Larger of one third of column dimension in direction of spacing or 200 mm for Rectangular column</td>
<td>200 mm</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Rectangular, larger of one third of column dimension in direction of spacing or 200 mm, spacing can be increased in centre of column when $h/b &gt; 20$</td>
<td>For DPRs and LDPRs: Larger of one-quarter of the column dimension (or diameter) in direction of spacing or 200 mm</td>
<td>For DPRs: Larger of one-quarter of the column dimension (or diameter) in direction of spacing or 200 mm</td>
<td></td>
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</tr>
</tbody>
</table>
--- | --- | --- | --- | --- | ---
**Maximum longitudinal column bar diameter** | For DPRs and LDPRs:  
\[
\frac{d_b}{h_b} \leq 3.2 \sqrt[3]{\frac{f_y}{f_c'}}
\]  
Bar diameter can be increased by 25% when plastic hinges are not expected to develop in column end zones and need not be met when bars remain in tension or compression over the length of the joint | - | - | 12.7 mm (minimum) | 50.8 mm (maximum) | 12.7 mm (minimum)

**Minimum diameter for transverse reinforcement (outside of the potential plastic hinge region)** | Rectangular hoops and ties  
5 mm for \(d_b < 20\)  
10 mm for \(20 \leq d_b < 32\)  
12 mm for \(d_b > 32\)  
Spiral or hoops of circular shape, 5 mm | - | Rectangular hoops and ties  
6 mm for \(d_b < 20\)  
10 mm for \(20 \leq d_b < 32\)  
12 mm for \(d_b > 32\)  
Spiral or hoops of circular shape, 6 mm | 6.35 mm | 6.35 mm > \(d_b/3\)

**Maximum vertical spacing of ties (outside of the potential plastic hinge region)** | Smaller of \(h_{min}/3\) or \(10d_b\) | Smaller of \(h_{min}/3\) or \(10d_b\) | If using \(\phi = 0.9\) smaller of \(h_{min}/5\) or \(16d_b\)  
If using \(\phi = 0.7\) smaller of \(h_{min}16d_b\) or \(48d_s\)  
For Spirally columns, \(d_c/6\)  
For tied columns, \(min(h_{min}/16d_b\) and \(48d_s)\) | For DPRs:  
It shouldn’t be lower than 70% of the ones within the plastic hinge region  
For DPRs:  
Smaller of \(2h_{min}/5\), \(12d_b\) or \(400\ mm\) | For tied columns, min \(\{12d_b, 1\ in.\ and \(2h_{min}/3\}\) | -

**Anti-buckling reinforcement (outside of the potential plastic hinge region)** | Rectangular hoops and ties  
\[
A_{te} = \frac{\sum A_b f_y s}{135 f_y t d_b}
\]  
Rectangular hoops and ties  
\[
A_{te} = \frac{\sum A_b f_y s}{135 f_y t d_b}
\]  
- | - | - | -
<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Confinement reinforcement (outside of the potential plastic hinge region)</td>
<td>Spirals or hoops of circular shape $\rho_s = \frac{A_{st}}{155d} \frac{f_y}{f_{yt}} \frac{1}{db}$</td>
<td>Spirals or hoops of circular shape $\rho_s = \frac{A_{st}}{155d} \frac{f_y}{f_{yt}} \frac{1}{db}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum shear reinforcement (outside of the potential plastic hinge region)</td>
<td>Rectangular hoops and ties $A_{sh}^3$</td>
<td>Rectangular hoops and ties $A_{sh}^9$</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Spirals or hoops of circular shape $\rho_s^4$</td>
<td>Spirals or hoops of circular shape $\rho_s^{10}$</td>
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<tr>
<td>Minimum diameter for transverse reinforcement (within potential plastic hinge region)</td>
<td>$A_v = \frac{1}{16} \sqrt{\frac{b_w S}{f_{ys}}}$</td>
<td>For DPRs and LDPRs: $A_v = \frac{1}{12} \sqrt{\frac{b_w S}{f_{ys}}}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum shear force (outside of the potential plastic hinge region)</td>
<td>$V_n \leq 0.2f'bd_w d, or 8 b_w d$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Minimum diameter for transverse reinforcement (within potential plastic hinge region)</td>
<td>Same as outside plastic hinge region</td>
<td>Same as outside plastic hinge region</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Maximum vertical spacing of ties (within potential plastic hinge region)</td>
<td>For DPRs: Smallest of $h_{min}/4$ or $6d_b$</td>
<td>For DPRs: Smallest of $h_{min}/4$ or $6d_b$</td>
<td>For DPRs: Smaller of $h/5$, diameter, $/5 6d_b$ or $200$ mm</td>
<td>Same as outside plastic hinge region</td>
<td>-</td>
</tr>
<tr>
<td>Anti-buckling reinforcement (within potential plastic hinge region)</td>
<td>For DPRs and LDPRs:</td>
<td>For DPRs:</td>
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</table>
--- | --- | --- | --- | --- | ---

**potential plastic hinge region)**

- For rectangular hoops and ties:
  \[ A_{te} = \frac{\sum A_{b, fy} S_h}{96 \cdot f_{yt} d_b} \]
- For spirals or hoops of circular shape:
  \[ \rho_s = \frac{A_{st}}{110d^2} \cdot \frac{f_y}{f_{yt}} \]

**Confinement reinforcement (within potential plastic hinge region)**

- For DPRs and LDPRs: Rectangular hoops and ties for DPRs, \( A_{sh}^{7} \) for LDPRs, 0.7 \( A_{sh}^{7} \)
  - Spirals or hoops of circular shape for DPRs, \( \rho_s^{8} \) for LDPRs, 0.7 \( \rho_s^{8} \)

**Minimum shear reinforcement (within potential plastic hinge region)**

- Same as outside plastic hinge region

**Maximum shear force (within potential plastic hinge region)**

- Same as outside plastic hinge region

### Note:

1. \( N_{n,max} = \alpha f_y (A_g - A_{st}) + f_y A_{st} \)
2. \( N_g^0 = 0.7 \phi N_{n,max} \)
3. \( A_{sh} = \frac{(1-\rho_s m) S_h K_{sh} A_g f_y}{A_{st} f_{yt}} \cdot \frac{N^*}{A_c f_{cy} \Phi f_y A_g} - 0.0065 S_h h' \) (\( N^* = \) design axial load at ultimate limit state)
4. \( \rho_s = \frac{(1-\rho_s m) A_g f_y}{A_{st} f_{yt} \Phi f_y A_g} - 0.0084 \) (\( N^* = \) design axial load at ultimate limit state)
5. \( DPR = \) Ductile Potential Plastic Region
<table>
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<tr>
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</thead>
<tbody>
<tr>
<td>6.</td>
<td>$LDPR = \text{Limited Ductile Potential Plastic Region}$</td>
<td></td>
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<tr>
<td>7.</td>
<td>$A_{sh} = \left(\frac{1.3-m}{3.3}\right)h\frac{A_g}{A_c} \frac{f_{c}'}{f_y} N^* - 0.006 S_h h^*$</td>
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<tr>
<td>8.</td>
<td>$\rho_s = \left(\frac{1.3-m}{2.4}\right)h\frac{A_g}{A_c} \frac{f_{c}'}{f_y} N^* - 0.0084$</td>
<td></td>
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<tr>
<td>9.</td>
<td>$A_{sh} = \left(\frac{1.3-m}{3.3}\right)h\frac{A_g}{A_c} \frac{f_{c}'}{f_y} N^* - 0.0065 S_h h^* \left(N^* = 0.85 \phi \alpha \frac{f_{c}'}{f_y} (A_g - A_{st}) + f_y A_{st}\right)$</td>
<td></td>
<td></td>
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<tr>
<td>10.</td>
<td>$\rho_s = \left(\frac{1.3-m}{2.4}\right)h\frac{A_g}{A_c} \frac{f_{c}'}{f_y} N^* - 0.0084 \left(N^* = 0.85 \phi \alpha \frac{f_{c}'}{f_y} (A_g - A_{st}) + f_y A_{st}\right)$</td>
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<tr>
<td>11.</td>
<td>$A_{sh} = 0.3 S_h h^* \left(\frac{A_g}{A_c} - 1\right) \frac{f_{c}'}{f_y}$</td>
<td></td>
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<tr>
<td>12.</td>
<td>$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1\right) \frac{f_{c}'}{f_y}$</td>
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<tr>
<td>13.</td>
<td>$P_e = \text{Maximum design axial load in compression at a given eccentricity}$</td>
<td></td>
<td></td>
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<tr>
<td>14.</td>
<td>$\rho_{s1} = 0.45 \left(\frac{A_g}{A_c} - 1\right) \frac{f_{c}'}{f_y} \left(0.5 + 0.125 \frac{P_e}{\phi f_{c}' A_g}\right)$</td>
<td></td>
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<tr>
<td>15.</td>
<td>$\rho_{s2} = 0.12 \frac{f_{c}'}{f_y} \left(0.5 + 0.125 \frac{P_e}{\phi f_{c}' A_g}\right)$</td>
<td></td>
<td></td>
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<tr>
<td>16.</td>
<td>$A_{sh} = 0.3 S_h h^* \left(\frac{A_g}{A_c} - 1\right) \frac{f_{c}'}{f_y} \left(0.5 + 1.25 \frac{P_e}{\phi f_{c}' A_g}\right)$</td>
<td></td>
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<tr>
<td>17.</td>
<td>$A_{sh} = 0.12 S_h h^* \frac{f_{c}'}{f_y} \left(0.5 + 1.25 \frac{P_e}{\phi f_{c}' A_g}\right)$</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>18.</td>
<td>$P_0 = \phi \left(0.85 f_{c}' (A_g - A_{st}) + A_{st} f_y\right)$</td>
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</table>
## C5D.4 Beam-Column Joint Reinforcement and Detailing

### Table C5D.6: Evolution of standards-based beam-column joints design/details requirements (Cuevas et al., 2015)

<table>
<thead>
<tr>
<th></th>
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</thead>
<tbody>
<tr>
<td>Maximum horizontal shear stress</td>
<td>$v_{jh} \leq \frac{(0.20f'_{c})}{10\text{MPa}}$</td>
<td>$v_{jh} \leq 0.25f'_{c}$</td>
<td>$v_{jh} \leq 0.25f'_{c}$</td>
</tr>
<tr>
<td>Minimum horizontal transverse confinement reinforcement</td>
<td>$\rho_s = \left(1 - \frac{p_t m}{A} \right) \frac{f'_{c}}{2.4} \frac{N'}{A\phi f_y} \frac{A_g}{A_c}$</td>
<td>$\rho_s = \frac{A_{st}}{155d^2 f_{y}}$</td>
<td>$\rho_s = 0.45 \left( \frac{A_g}{A_c} - 1 \right) \frac{f'<em>{c}}{f</em>{y_h}}$</td>
</tr>
<tr>
<td>For spirals or circular hoops:</td>
<td>$\rho_s = \frac{A_{st}}{155d^2 f_{y}}$</td>
<td>$\rho_s = \frac{A_{st}}{155d^2 f_{y}}$</td>
<td>$\rho_s = 0.45 \left( \frac{A_g}{A_c} - 1 \right) \frac{f'<em>{c}}{f</em>{y_h}}$</td>
</tr>
<tr>
<td>$A_{sh} = \frac{(1 - p_t m) s_h h' A_g f' c}{3.3 A_c f_y f' c A_g}$</td>
<td>$A_{sh} = \frac{(1 - p_t m) s_h h' A_g f' c}{3.3 A_c f_y f' c A_g}$</td>
<td>$A_{sh} = \frac{(1 - p_t m) s_h h' A_g f' c}{3.3 A_c f_y f' c A_g}$</td>
<td>$A_{sh} = 0.3s_h h' \frac{A_g}{A_c} \frac{f'<em>{c}}{f</em>{y_h}}$</td>
</tr>
<tr>
<td>With $\left{ \frac{A_g}{A_c} \leq 1.50, \frac{p_t m}{A} \leq 0.40 \right}$</td>
<td>With $\left{ \frac{A_g}{A_c} \leq 1.20, \frac{p_t m}{A} \leq 0.40 \right}$</td>
<td>With $\left{ \frac{A_g}{A_c} \leq 1.20, \frac{p_t m}{A} \leq 0.40 \right}$</td>
<td>With $\left{ \frac{A_g}{A_c} \leq 1.20, \frac{p_t m}{A} \leq 0.40 \right}$</td>
</tr>
<tr>
<td>(reduce by half when joints connecting beams at all four column faces)</td>
<td>(reduce by half when joints connecting beams at all four column faces)</td>
<td>(reduce by half when joints connecting beams at all four column faces)</td>
<td>(reduce by half when joints connecting beams at all four column faces)</td>
</tr>
<tr>
<td>Spacing limits</td>
<td>$S_{max} = \left( \frac{(D, b, h)}{10d_b} \right) \frac{3}{200 \text{ mm}}$</td>
<td>$S_{max} = \left( \frac{(D, b, h)}{10d_b} \right) \frac{3}{200 \text{ mm}}$</td>
<td>$S_{max} = \left( \frac{(D, b, h)}{10d_b} \right) \frac{3}{200 \text{ mm}}$</td>
</tr>
<tr>
<td>Design yield strength (for earthquake)</td>
<td>$f_{y_h} \leq 500\text{MPa}$</td>
<td>$f_{y_v} \leq 500\text{MPa}$</td>
<td>$f_{y_h} \leq 500\text{MPa}$</td>
</tr>
<tr>
<td>Maximum horizontal shear stress (for earthquake)</td>
<td>$v_{jh} \leq \frac{(0.20f'_{c})}{10\text{MPa}}$</td>
<td>$v_{jh} \leq 0.25f'_{c}$</td>
<td>$v_{jh} \leq 1.5\sqrt{f'_{c}}$</td>
</tr>
<tr>
<td>-------------</td>
<td>---------------</td>
<td>---------------</td>
<td>---------------</td>
</tr>
<tr>
<td>Minimum horizontal joint reinforcement (for earthquake)</td>
<td>For spirals or circular hoops: [ \rho_s = \frac{A_{st}}{110d^2} \frac{f_y}{f_y} \frac{d_b}{200} ]</td>
<td>For spirals or circular hoops: [ \rho_s = 0.70 \left( \frac{A_{st}}{110d^2} \frac{f_y}{f_y} \frac{d_b}{200} \right) ]</td>
<td>For spirals and circular hoops, the greater of: [ \rho_s = 0.45 \frac{A_g}{A_c} - 1 \frac{f'_c}{f_y} \frac{0.5 + 1.25}{\phi f'_c A_g} ]</td>
</tr>
<tr>
<td></td>
<td>For rectangular hoop and tie reinforcement: [ A_{sh} = \frac{1}{3}h^* A_g \frac{f'_c}{f_y} \frac{N'}{f'_c A_g} - 0.006s_b h^* ]</td>
<td>For rectangular hoop and tie reinforcement: [ A_{sh} = 0.70 \left( \frac{1}{3}h^* A_g \frac{f'_c}{f_y} \frac{N'}{f'_c A_g} - 0.006s_b h^* \right) ]</td>
<td>For rectangular hoop and tie reinforcement, the greater of: [ A_{sh} = 0.3s_b h^* \left( \frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_y} \frac{0.5 + 1.25}{\phi f'_c A_g} ]</td>
</tr>
<tr>
<td></td>
<td>With [ A_g/A_c \leq 1.50 ] [ p_{tm} \leq 0.40 ] [ (\ast) 70% \text{ reduction for limited ductile} ]</td>
<td>With [ A_g/A_c \leq 1.20 ] [ p_{tm} \leq 0.40 ] [ (\ast) 70% \text{ reduction not allowed at the joint of the columns of the first storey} ]</td>
<td></td>
</tr>
<tr>
<td>Spacing limits (for earthquake)</td>
<td>[ S_{max} = \begin{cases} \frac{(D, b, h)}{4} &amp; \text{(ductile)} \ 6d_b &amp; \text{200 mm} \end{cases} ]</td>
<td>[ S_{max} = \begin{cases} \frac{(D, b, h)}{4} &amp; \text{(ductile)} \ 6d_b &amp; \text{200 mm} \end{cases} ]</td>
<td>[ S_{max} = \begin{cases} \frac{(D, b, h)}{5} &amp; \text{(ductile)} \ 6d_b &amp; \text{200 mm} \end{cases} ]</td>
</tr>
<tr>
<td></td>
<td>[ S_{max} = \begin{cases} \frac{(D, b, h)}{4} &amp; \text{(limited ductile)} \ 10d_b &amp; \text{200 mm} \end{cases} ]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing limits for vertical reinforcement (for ductile members adjacent to the joint)</td>
<td>[ S_{max} = \begin{cases} \frac{(D, h, b)}{4} &amp; \text{(at least one intermediate bar in each side of the column in that plane)} \ 200 \text{ mm} \end{cases} ]</td>
<td>[ S_{max} = \begin{cases} \frac{(D, h, b)}{4} &amp; \text{(at least one intermediate bar in each side of the column in that plane)} \ 200 \text{ mm} \end{cases} ]</td>
<td>[ S_{max} = 200 \text{ mm} ]</td>
</tr>
<tr>
<td></td>
<td>[ (at least one intermediate bar in each side of the column in that plane) ]</td>
<td>[ (at least one intermediate bar in each side of the column in that plane) ]</td>
<td>[ (at least one intermediate bar in each side of the column in that plane) ]</td>
</tr>
</tbody>
</table>
### Requirement

<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>Maximum diameter of longitudinal beam bars passing through joints (for ductile members adjacent to the joint)</td>
<td>$\frac{d_b}{h_c} \leq 3.3 \alpha_d \sqrt{\frac{f'_c}{1.25f_y}}$</td>
<td>$f'_c \leq 70\text{MPa}$</td>
<td>$\alpha_d = 1.00$ (ductile) $\alpha_d = 1.20$ (limited ductile)</td>
</tr>
</tbody>
</table>

### Maximum diameter of column bars passing through joints (for ductile members adjacent to the joint)

<table>
<thead>
<tr>
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</thead>
<tbody>
<tr>
<td>$\frac{d_b}{h_b} \leq 3.2 \frac{\sqrt{f_c}}{f_y}$ (1)</td>
<td>$\frac{d_b}{h_b} \leq 4.0 \frac{\sqrt{f_c}}{f_y}$ (2)</td>
<td>For columns designed by Method B or by Method A (and the joint is below the mid-height of the second storey)</td>
<td>For columns designed by Method A and the joint is above the mid-height of the second storey</td>
</tr>
</tbody>
</table>

### Note:

NZS 3101P:1970, clause 1.2.6 states that “…The reinforcing spiral shall extend from the floor level in any storey or from the top of the footing to the level of the lowest horizontal reinforcement in the slab, drop panel, or beam above.” Therefore, no spiral, hoop or tie is required in the beam-column joint.
C5D.5 Wall Reinforcement and Detailing

Table C5D.7 summarises the evolution of the New Zealand standards-based design/details requirements for walls, while Table C5D.8 provides a key to the notation used throughout the various Standards.

Table C5D.7: Evolution of standards-based design/details requirements for walls

<table>
<thead>
<tr>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum thickness-general</td>
<td>100 mm</td>
<td>100 mm for the uppermost 4m of wall height and for each successive 7.5 m downward (or fraction thereof), shall be increased by 25 mm</td>
<td>150 mm for the uppermost 4m of wall height and for each successive 7.5 m downward (or fraction thereof), shall be increased by 25 mm</td>
<td>6 in.</td>
<td>5 in.</td>
</tr>
</tbody>
</table>
| Limitations on the height to thickness ratio | If $N^* > 0.2f'_cA_g$  
$k_eL_n \leq 30$  
$L_n$: the clear vertical distance between floors or other effective horizontal lines of lateral support | If $N^* > 0.2f'_cA_g$  
$L_n \leq 25$  
UNLESS:  
1- the neutral axis depth for the design loading $\leq 4b$ or $0.3l_w$  
2- Any part of the wall within a distance of $3b$ from the inside of a continuous line of lateral support provided by a flange or cross wall | $L_n \leq 10$  
$L_n$: the distance between lateral supports (Horizontal or Vertical) | $L_n \leq 35$  
$L_n$: the distance between lateral supports (Horizontal or Vertical) | $L_n \leq 24$  
$L_n$: the distance between lateral supports (Horizontal or Vertical) |
| Singly reinforced walls              | No limitations | No limitations | No limitations | No limitations | No limitations |
| Limitations on the height to thickness ratio to prevent flexural torsional buckling of in-plane loaded walls | $k_rL_n \leq 12 \sqrt{\frac{l_n}{l_w}}$  
where:  
$N^* \leq 0.015f'_cA_g$  
and $\frac{j_n}{t} \leq 75$  
and $\frac{k_nL_n}{t} \leq 65$ | | | | |
<table>
<thead>
<tr>
<th>Doubly reinforced walls</th>
<th>No requirements</th>
<th>No requirements</th>
<th>No requirements</th>
<th>No requirements</th>
<th>No requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Moment magnification required when:</strong></td>
<td></td>
<td></td>
<td></td>
<td>No requirements</td>
<td>No requirements</td>
</tr>
</tbody>
</table>
|  | \[ b_m = \frac{\alpha k_m \beta (A_r + 2) L_w}{1700 \sqrt{f'_c}} \]  
\[ \beta = 7 \text{ (DFR)} \]  
\[ \beta = 5 \text{ (LDPR)} \]  | \[ b_m = \frac{k_m (\mu + 2) (A_r + 2) L_w}{1700 \sqrt{f'_c}} \]  |  | No requirements |  |
| **Ductile detailing length - special shear stress limitations** | \[ \max \left\{ L_w, 0.17 \frac{M}{f_y} \right\} \]  
Measured from the 1st flexural yielding section  
Need not be greater than \( 2L_w \) | \[ \max \left\{ L_w, \frac{h_w}{6} \right\} \]  
Measured from the 1st flexural yielding section  
Need not be greater than \( 2L_w \) | \[ \max \left\{ L_w, \frac{h_w}{6} \right\} \]  
Measured from the 1st flexural yielding section  
Need not be greater than \( 2L_w \) | No requirements | No requirements |
| **Limitation on the use of singly reinforced walls** | \( \rho_l \leq 0.01 \)  
\( b \leq 200 \text{ mm} \)  
\( \mu \leq 4 \) | \( b \leq 200 \text{ mm} \)  
\( \mu \leq 4 \)  
\( b \leq 200 \text{ mm} \) or if the design shear stress \( \leq 0.3 \frac{f'_c}{L_w} \) |  
Earth retaining walls:  
\( b < 10 \text{ in.} \)  
Other walls:  
\( b < 9 \text{ in.} \) |  
\( t < 10 \text{ in.} \) |  |
| **Minimum longitudinal reinforcement ratio** | \( \rho_n = \frac{\sqrt{f'_c}}{4 f_y} \) | \( \rho_l = \frac{0.7}{f_y} \) | \( \rho_l = \frac{0.7}{f_y} \) | \( \frac{9000}{f_y} \% \geq 0.18\% \)  
Note: \( f_y \) in units of [psi]  
\( 0.0025 \) (mild steel)  
\( 0.0019 \) (high tensile steel) |  |
<table>
<thead>
<tr>
<th><strong>Maximum longitudinal reinforcement ratio (( \rho_l ))</strong></th>
<th>( \frac{16}{f_y} )</th>
<th>( \frac{16}{f_y} )</th>
<th>( \frac{16}{f_y} )</th>
<th>No requirements</th>
<th>No requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum spacing of longitudinal reinforcement</td>
<td>Min ( {Lw/3, 3t, \text{or} 450 \text{ mm}} )</td>
<td>Min ( {2.5b, 450 \text{ mm}} )</td>
<td>Min ( {2.5b, 450 \text{ mm}} )</td>
<td>Min ( {2.5b, 18\text{ in.} \text{ (457 mm)}} )</td>
<td>2.5b</td>
</tr>
<tr>
<td>Anti-buckling reinforcement (Outside of the</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No requirements</td>
</tr>
<tr>
<td>potential plastic hinge region)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No requirements</td>
</tr>
<tr>
<td>Where: ( \rho_l &gt; \frac{2}{f_y} ) (\frac{DPR}{3} ) (\frac{LDPR}{f_y}) (d_{tie} &gt; d_b/4) (\text{Spacing &lt; 12}d_b)</td>
<td>( \rho_l &gt; \frac{2}{f_y} ) (\frac{DPR}{3} ) (\frac{LDPR}{f_y}) (d_{tie} &gt; d_b/4) (\text{Spacing &lt; 12}d_b)</td>
<td>( \rho_l &gt; \frac{2}{f_y} ) (\frac{DPR}{3} ) (\frac{LDPR}{f_y}) (d_{tie} &gt; d_b/4) (\text{Spacing &lt; 12}d_b)</td>
<td>No requirements</td>
<td>No requirements</td>
<td></td>
</tr>
<tr>
<td>Anti-buckling reinforcement (Within the</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No requirements</td>
</tr>
<tr>
<td>potential plastic hinge region)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No requirements</td>
</tr>
<tr>
<td>Where: ( \rho_l &gt; \frac{2}{f_y} ) (\frac{DPR}{3} ) (\frac{LDPR}{f_y}) (A_{te} = \sum A_{bf} f_y s \frac{96}{f_{yt} d_b}) (\text{Spacing} \leq \left{ \begin{array}{l} 6d_b \text{ (DPR)} \text{Spacing} \leq 6d_b \end{array} \right.)</td>
<td>( \rho_l &gt; \frac{2}{f_y} ) (\frac{DPR}{3} ) (\frac{LDPR}{f_y}) (A_{te} = \sum A_{bf} f_y s \frac{96}{f_{yt} d_b}) (\text{Spacing} \leq \left{ \begin{array}{l} 6d_b \text{ (DPR)} \text{Spacing} \leq 6d_b \end{array} \right.)</td>
<td>( \rho_l &gt; \frac{2}{f_y} ) (\frac{DPR}{3} ) (\frac{LDPR}{f_y}) (A_{te} = \sum A_{bf} f_y s \frac{96}{f_{yt} d_b}) (\text{Spacing} \leq \left{ \begin{array}{l} 6d_b \text{ (DPR)} \text{Spacing} \leq 6d_b \end{array} \right.)</td>
<td>No requirements</td>
<td>No requirements</td>
<td></td>
</tr>
<tr>
<td>Confinement reinforcement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No requirements</td>
</tr>
<tr>
<td>Where neutral axis depth</td>
<td>( &gt; c_c = \frac{0.3 \phi_c L_w}{\lambda} ) (\lambda = 1.0 \text{ (DPR)} ) (\lambda = 2.0 \text{ (LDPR)}) (A_{sh} = \alpha s_h h'' \frac{A_g f_c'}{A_c f_y h} \left( \frac{c}{L_w} \right)^{-0.07}) (\alpha = 0.25 \text{ (DPR)} ) (\alpha = 0.175 \text{ (LDPR)})</td>
<td>( &gt; c_c = \frac{0.3 \phi_c L_w}{\lambda} ) (\lambda = 1.0 \text{ (DPR)} ) (\lambda = 2.0 \text{ (LDPR)}) (A_{sh} = \alpha s_h h'' \frac{A_g f_c'}{A_c f_y h} \left( \frac{c}{L_w} \right)^{-0.07}) (\alpha = 0.25 \text{ (DPR)} ) (\alpha = 0.175 \text{ (LDPR)})</td>
<td>( &gt; c_c = \frac{0.3 \phi_c L_w}{\lambda} ) (\lambda = 1.0 \text{ (DPR)} ) (\lambda = 2.0 \text{ (LDPR)}) (A_{sh} = \alpha s_h h'' \frac{A_g f_c'}{A_c f_y h} \left( \frac{c}{L_w} \right)^{-0.07}) (\alpha = 0.25 \text{ (DPR)} ) (\alpha = 0.175 \text{ (LDPR)})</td>
<td>No requirements</td>
<td>No requirements</td>
</tr>
<tr>
<td>------------------------------------------</td>
<td>--------------------</td>
<td>--------------------</td>
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<td>-----------------------</td>
</tr>
<tr>
<td>Maximum spacing of confinement reinforcement</td>
<td>DPR: ( \min {6d_b, 0.5t} )</td>
<td>( \min {6d_b, 0.5t, 150 \text{ mm}} )</td>
<td>( \min {6d_b, 0.5t, 150 \text{ mm}} )</td>
<td>No requirements</td>
<td>No requirements</td>
</tr>
<tr>
<td></td>
<td>LDPR: ( \min {10d_b, t} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum confinement length</td>
<td>( \max \left{ \left( \frac{c - 0.7c_c}{c} \right), \frac{0.5c}{c} \right} )</td>
<td>( \max \left{ \left( \frac{c - 0.7c_c}{c} \right), \frac{0.5c}{c} \right} )</td>
<td>( 0.5c )</td>
<td>No requirements</td>
<td>No requirements</td>
</tr>
<tr>
<td>Maximum nominal shear stress</td>
<td>( v_n \leq 0.2f'_c ) or 8MPa</td>
<td>( v_n \leq \left{ \begin{array}{l} 0.2f'_c \ 1.1\sqrt{f'_c} \ 9\text{MPa} \end{array} \right} )</td>
<td>( v_n \leq 0.2f'_c ) or 6MPa</td>
<td>( v_u )</td>
<td>( v = \frac{f_c}{1 + \frac{h^2}{49t^2}} )</td>
</tr>
<tr>
<td>Concrect shear strength (simplified)</td>
<td>( V_c = \min \left{ \begin{array}{l} 0.17\sqrt{f'<em>c}A</em>{cv} \ 0.17\left[ f'<em>c + \frac{4.9}{A_p}\right]A</em>{cv} \end{array} \right} )</td>
<td>( V_c = \min \left{ \begin{array}{l} 0.2\sqrt{f'_c} \ 0.2\left[ f'_c + \frac{N}{A_p}\right] \end{array} \right} )</td>
<td>( V_c = \min \left{ \begin{array}{l} 0.2\sqrt{f'_c} \ 0.2\left[ f'_c + \frac{P_u}{A_p}\right] \end{array} \right} )</td>
<td>The shear stress carried by the concrete shall not exceed:</td>
<td>No requirements</td>
</tr>
<tr>
<td>Shear reinforcement</td>
<td>( A_v = \frac{V_n}{f_{yt}} )</td>
<td>( A_v = \frac{(v_n - v_c)b_w s_2}{f_{yt}} )</td>
<td>( A_v = \frac{(v_n - v_c)b_w s_2}{f_{yh}} )</td>
<td>( A_v = \frac{V'_u}{\phi f_c d \left( \frac{H}{D} - 1 \right)} )</td>
<td>No requirements</td>
</tr>
<tr>
<td>-------------------------------------------------</td>
<td>---------------</td>
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<td>-----------------------</td>
</tr>
<tr>
<td><strong>Minimum shear reinforcement</strong></td>
<td>$A_v = \frac{0.7 , b_w , s_2}{f_{yt}}$</td>
<td>$A_v = \frac{0.7 , b_w , s_2}{f_{yt}}$</td>
<td>$A_v = \frac{0.7 , b_w , s_2}{f_{yh}}$</td>
<td>$A_v = \frac{v_{ix} , d}{\phi , f_{yd}}$ or $\frac{v_{ix} , d}{\phi , f_{yd}} \geq 0.18$</td>
<td>0.0025 (mild steel) 0.0018 (high tensile steel)</td>
</tr>
<tr>
<td><strong>Maximum spacing of shear reinforcement</strong></td>
<td>$\text{Min}\left(\frac{l_w}{5}, 3t, \text{or } 450 , \text{mm}\right)$</td>
<td>$\text{Min}\left(\frac{l_w}{5}, 3t, \text{or } 450 , \text{mm}\right)$</td>
<td>$\text{Min}\left(\frac{l_w}{5}, 3t, \text{or } 450 , \text{mm}\right)$</td>
<td>$2.5t$</td>
<td>2.5t</td>
</tr>
<tr>
<td><strong>Vertical reinforcement</strong></td>
<td>$\rho_n \geq \frac{0.7}{f_{yn}}$</td>
<td>$\rho_n \geq \frac{0.7}{f_{yn}}$</td>
<td>$\rho_n \geq \frac{0.7}{f_{yn}}$</td>
<td>No requirements</td>
<td>No requirements</td>
</tr>
<tr>
<td><strong>Maximum shear strength provided by the concrete in ductile detailing length</strong></td>
<td>$V_c = \left(0.27 \lambda \sqrt{f_c} + \frac{N^*}{4A_g}\right)b_wd \geq 0.0$</td>
<td>$V_c$ shall not be taken larger than: $v_c = 0.6 \frac{N^*}{A_g}$</td>
<td>$V_c$ shall not be taken larger than: $v_c = 0.6 \frac{f_c}{A_g}$</td>
<td>No requirements</td>
<td>No requirements</td>
</tr>
<tr>
<td><strong>Splicing of flexural tension reinforcement</strong></td>
<td>One-third ($DPR$) and one-half ($LDPR$) of reinforcement can be spliced where yielding can occur</td>
<td>One-third of reinforcement can be spliced where yielding can occur</td>
<td>One-third of reinforcement can be spliced where yielding can occur</td>
<td>One-half of reinforcement can be spliced where yielding can occur</td>
<td>No requirements</td>
</tr>
<tr>
<td>-------------------------------------------------</td>
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<td>-----------------------</td>
</tr>
</tbody>
</table>
| Maximum compressive stress in concrete          | No requirements | No requirements | No requirements | $\left[1 - \left(\frac{h}{3.5d}\right)^3\right]0.2f'_c$ | Direct loading: $k_{f_{cu}}$  
$k = \frac{p}{\frac{5}{12}} - 0.007\frac{h}{t} + 0.2$  
$f_{cu}$: minimum crushing strength  
$p$: total percentage of vertical reinforcement  
$0.25 \leq p \leq 0.5$  
$\frac{h}{t} \geq 10$  
Seismic bending + direct stress:  
$1.25k$ |
| Maximum stress in the tensile steel             | No requirements | No requirements | No requirements | No requirements | 15000 psi for mild steel  
20000 psi for the special types of reinforcement covered by the First Schedule hereto |

**Note:**  
NZS 3101P:1970 units of [psi]
Table C5D.8: Notation used in New Zealand standards for walls

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</thead>
<tbody>
<tr>
<td>Design axial load at the ultimate limit state</td>
<td>$N^*$</td>
<td>$N^*$</td>
<td>$P_u$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>The clear vertical distance between floors or other effective horizontal lines of lateral support, or clear span</td>
<td>$L_n$</td>
<td>$L_n$</td>
<td>$L_n$</td>
<td>$h$</td>
<td>$h$</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>$t, b$</td>
<td>$b$</td>
<td>$b$</td>
<td>$d, b$</td>
<td>$t$</td>
</tr>
<tr>
<td>Effective length factor for Euler buckling</td>
<td>$k_e$</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Effective length factor for flexural torsional buckling</td>
<td>$k_{ft}$</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Horizontal length of wall</td>
<td>$L_w$</td>
<td>$l_w$</td>
<td>$l_w$</td>
<td>D</td>
<td>N/A</td>
</tr>
<tr>
<td>Thickness of boundary region of wall at potential plastic hinge region</td>
<td>$b_m$</td>
<td>$b_m$</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Total height of wall from base to top</td>
<td>$h_w$</td>
<td>$h_w$</td>
<td>$h_w$</td>
<td>$H$</td>
<td>N/A</td>
</tr>
<tr>
<td>Aspect ratio of wall ($h_w/L_w$)</td>
<td>$A_r$</td>
<td>$A_r$</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Yield strength of non-prestressed reinforcement</td>
<td>$f_y$</td>
<td>$f_y$</td>
<td>$f_y$</td>
<td>$f_y$</td>
<td>N/A</td>
</tr>
<tr>
<td>Yield strength of transverse reinforcement</td>
<td>$f_{yh}$</td>
<td>$f_{yh}$</td>
<td>$f_{yh}$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Yield strength of shear reinforcement</td>
<td>$f_{yt}$</td>
<td>$f_{yt}$</td>
<td>$f_{yh}$</td>
<td>$f_y$</td>
<td>N/A</td>
</tr>
<tr>
<td>Ratio of vertical (longitudinal) wall reinforcement area to gross concrete area of horizontal section</td>
<td>$\rho_n = \frac{A_t}{A_g^*}$</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>The ratio of vertical wall reinforcement area to unit area of horizontal gross concrete section</td>
<td>$\rho_l = \frac{A_s}{t_{sv}}$</td>
<td>$\rho_l = \frac{A_s}{b_{sv}}$</td>
<td>$\rho_l = \frac{A_s}{b_{sv}}$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Diameter of longitudinal bar</td>
<td>$d_b$</td>
<td>$d_b$</td>
<td>$d_b$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Centre-to-centre spacing of shear reinforcement along member</td>
<td>$s$</td>
<td>$s$</td>
<td>$s$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Computed distance of neutral axis from the compression edge of the wall section</td>
<td>$c$</td>
<td>$c$</td>
<td>$c$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>A limiting depth for calculation of the special transverse reinforcement</td>
<td>$c_c$</td>
<td>$c_c$</td>
<td>$c_c$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Overstrength factor</td>
<td>$\phi_{uw}$</td>
<td>$\phi_o$</td>
<td>$\phi_o$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Area of concrete core</td>
<td>$A_c^*$</td>
<td>$A_c^*$</td>
<td>$A_c^*$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Gross area of concrete section</td>
<td>$A_g^*$</td>
<td>$A_g^*$</td>
<td>$A_g^*$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
### Notation

| Dimension of concrete core of rectangular section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop | $h''$ | $h''$ | $h''$ | N/A | N/A |
| Centre-to-centre spacing of hoop sets | $s_h$ | $s_h$ | $s_h$ | N/A | N/A |
| Structural type factor | ---- | ---- | $s$ | N/A | N/A |
| Displacement ductility capacity relied on in the design | N/A | $\mu$ | N/A | N/A | N/A |
| Area used to calculate shear area | $A_{cv}$ | N/A | N/A | N/A | N/A |
| Total nominal shear strength | $V_n$ | $V_n$ | $V_n$ | N/A | N/A |
| Design shear force | $V^*$ | $V^*$ | $V_u$ | $V_u$ | N/A |
| Concrete shear strength | $V_c$ | N/A | N/A | N/A | N/A |
| Nominal shear strength provided by shear reinforcement | $V_s$ | N/A | N/A | N/A | N/A |
| Shear stress provided by concrete | $v_c$ | $v_c$ | $v_c$ | $v_c$ | N/A |
| Centre-to-centre spacing of horizontal shear reinforcement | $s2$ | $s2$ | $s2$ | $s$ | N/A |
Appendix C5E: Diaphragm Grillage Modelling Methodology

C5E.1 Assessment Approach

For buildings that are essentially rectangular with relatively uniform distribution of vertical lateral force resisting systems across the plan of the building, and no significant change of plan with height, simple, hand-drawn strut and tie solutions can be used.

However, buildings with significant asymmetry in the location of lateral force resisting elements (distribution across the building plan, termination up the height of the building, varying stiffness and/or strength between vertical elements) may require a more sophisticated analysis. For these types of structures, a grillage method can be used to obtain diaphragm design actions. Details of a simple grillage method appropriate for design office use are given below (Holmes, 2015).

C5E.2 Grillage Section Properties

Grillage members are typically modelled as concrete elements, without reinforcement modelled, in an elastic analysis program. Figure C5E.1 illustrates a grillage model developed for a complicated podium diaphragm.

![Example of a grillage model for podium diaphragm (Holmes, 2015)](image)

The recommended dimensions of the grillage elements for the modelling of a flat plate are based on work completed by Hrennikoff (1941), as shown in Figure C5E.2. This solution is based on a square grillage (with diagonal members). Rectangular grillages can also be used; the dimensions of the grillage beams will vary from those given for the square grillage solution (Hrennikoff, 1941).
Floors can be assumed to be uncracked for the purposes of diaphragm assessments. Given that diaphragms typically contain low quantities of longitudinal reinforcing steel and considering transformed section effects, it is not considered necessary to include longitudinal reinforcement when determining grillage section properties. An exception to this is the determination of the section properties for collector elements.

It is recommended that the effective stiffness of collector elements is based on the transformed section of the concrete plus:

- the bars reinforcing the collector element, or
- the structural steel beam acting in a collector.

Typically, when a collector is stretched and the strain in the steel approaches the yield strain, there will be significant cracking of the concrete that contributes to the collector. The effective stiffness of the collector, in tension, will reduce. However, for the typical steel contents of collector elements this reduction in stiffness is relatively small.

**Note:**
The collector is also typically required to resist compression forces due to the cyclic nature of seismic loading. Therefore, for modelling the collector element it is generally satisfactory to use either the transformed section of concrete and steel or the steel without the concrete. The combined concrete and steel option is stiffer than the steel-only option, so will attract more force.

### C5E.3 Effective Width of Grillage Members

The recommended effective grillage member widths for orthogonal and diagonal members are as follows (Hrennikoff, 1941):

- **Orthogonal members:**
  - width $A = 0.75 \times$ grid spacing
  - carries both tension and compression forces
- **Diagonal members:**
  - width $B = 0.53 \times$ grid spacing
  - carries compression forces only.
C5E.4 Effective Thickness of Grillage Members

The recommended thickness of the grillage beams depends on the floor construction as follows:

- **Hollow-core and Tee units:**
  - parallel to the units: average thickness (per metre width) to match the combined areas of the topping plus unit
  - perpendicular to the units: the average thickness (per metre width) of the combined areas of the topping and the top flange of the units

- **Rib and timber in-fill:**
  - parallel to the ribs: average thickness (per metre width) of combined areas of the topping and ribs
  - perpendicular to the ribs: average thickness (per metre width) of the topping only

- **In situ slabs and flat slabs:**
  - combined thickness of the topping and units (if present) parallel and transverse to the units (if present)

- **Steel profile composite floors:**
  - parallel to the webs: average of cross-section flange and web
  - transverse to the webs: thickness of the flange

- **Spaced hollow-core units with in situ slabs:**
  - following the concepts above, the designer should rationalise the effective thickness, parallel and perpendicular to the units.

C5E.5 Spacing of Grillage Members

It is recommended that a grillage beam spacing of 1.0 m is typically adequate to produce reasonable distribution of forces (Gardiner, 2011). It is advisable to try larger and smaller grid spacings to determine if the model is sufficiently refined.

In general terms, the point of sufficient refinement for the grid spacing is when the actions reported in the beams of the grillage change very little from the previous trial.

In order to get a desirable, higher resolution of forces, grillage spacings should be reduced while maintaining the square format (divide the main square grillage into sets of smaller squares) for the following situations:

- **Around the nodes where vertical structures (e.g. beams, columns, walls and eccentrically braced frames (EBFs)) would be connected to the floor plate.** This applies to vertical elements, both on the perimeter of the floor as well as within the interior of the floor:
  - internal frames
  - frames, walls or EBFs, etc. next to floor penetrations (typically stairs, escalators and lifts)

- **Around floor penetrations (typically stairs, escalators and lifts)**

- **At re-entrant corners in the floor plate**
Part C – Detailed Seismic Assessment

- For collectors, smaller sets of square grillages may be used either side of a collector (a grillage member with properties relevant to the collector performance). If a collector is relatively wide (say, greater than half the typical grillage spacing) consider modelling the collector as a small grillage/truss along the length of a collector, with the smaller set of squares either side of this.

**C5E.6 Supports, Nodes and Restraint Conditions**

The grillage is set up as a framework of struts. The junctions of the strut grillage framework are called “nodes”. Floor inertia loads will typically be applied to all of the nodes of the grillage. Each vertical structural element will be associated with one or more nodes in the grillage as follows:

- Columns – typically a single node
- Walls – typically a number of nodes along the length of the wall.

The vertical translational degree of freedom of nodes which coincide with vertical structural elements (i.e. columns or wall elements) should be fixed. The horizontal translational degrees of freedom of these nodes should be left unrestrained. The reasons for this are as follows:

- Forces going in to or out of the nodes associated with the vertical elements are in equilibrium with the inertia and transfer or deformation compatibility forces within the floor plate.
- If the horizontal degree of freedom was fixed, the loads applied to these nodes would go directly to the support point and would not participate in the force distribution of the floor plate.
- Transfer or deformation compatibility forces are internal forces and must balance at the vertical supports and across the floor plate.

**Note:**

If all of the horizontal degrees of freedom are left unrestrained in a computer analysis model the analysis will not run. Therefore, it is recommended that two nodes are fixed; with both horizontal degrees of freedom fixed at one node and with fixity only in the direction of the applied inertia at the second node (i.e. free to move in the perpendicular direction).

**C5E.7 Loss of Load Paths due to Diaphragm Damage**

Modify the grillage to account for anticipated diaphragm damage. For example, where floor to beam separation similar to that illustrated in Figure C5E.3 is anticipated due to beam elongation, the diagonal strut in the grillage should be removed recognising that the compression struts may not be able to traverse the damaged area (refer also to Figure C5E.3).
C5E.8 Application of Inertia Forces Introduced into the Grillage Model

Inertia of the floor, determined from pseudo-Equivalent Static Analysis (pESA) (refer to Section C2), is distributed over the framework of grillage elements, at the nodes of the orthogonal members of the grillage and in accordance to the tributary mass at each node:

- Tributary mass attributed to each node will include the seismic mass of the floor and any of the vertical structures attached to that node or nodes of the floor (i.e. walls, columns, beams, braces etc.).

- As a result of the “weighted” distribution of inertia associated with the appropriate mass attributed to each node, the distribution of inertia will not be uniform across the floor. There are concentrations of mass at frame lines, for example (beams, columns and cladding), and a more even distribution of inertia over the floor areas.

- Note that no inertia is placed where the diagonal member cross, because there is no node where the diagonal members pass. The diagonal members run between the nodes of the orthogonal grillage.

Inertia forces, applied to the structure, will be balanced by the forces at the supports/nodes of the floor plate. Other “internal” forces that balance the remaining portion of the forces at supports/nodes arise from deformation compatibility between the vertical structural systems being constrained to similar lateral displaced shapes. The largest of these compatibility forces are traditionally called “transfer” forces. Deformation compatibility forces occur in all buildings on all floors to varying degrees. All forces, applied and internal, must be in equilibrium.
C5E.9 Application of “Floor Forces”

Forces entering or leaving the floor where the floor is connected to the vertical lateral force resisting structures have been called “floor forces”, \( F_{Di} \). Floor forces can be determined from the results of the pESA (refer to Section C2) and, as illustrated in Figure C5, are equal to the difference in shears in vertical lateral load resisting elements above and below the diaphragm being assessed.

\[
F_{Di+1} = V_{Di+1} - V_{Di-1}
\]

**Figure C5:** Floor forces, \( F_{Di} \), determined from pESA (Holmes, 2015)

It is important that members of the vertical lateral force resisting systems in the pESA analysis model have in-plane and out-of-plane stiffness and that the analysis model has been enabled to report both major and minor axis actions of vertical elements.

Outputs for such elements should report actions in the X and Y directions. Therefore, for a given direction of earthquake attack, at each node there will be forces to be applied in the X and Y directions (refer to Figure C5E.5). Care is required to ensure that sign conventions (i.e. input and output of actions) are maintained.

**Figure C5E.5:** Floor forces \( F_{Di} \) in both X and Y directions at nodes connected to vertical elements – for one direction of earthquake attack (Holmes, 2015)
C5E.10 Out-of-Plane Push and Pull of Vertical Elements

Vertical elements (i.e. walls, columns, braced frames) are pushed out of plane at some stage during a seismic event. Depending on the magnitude of the inter-storey drift demands, these elements may yield, exhibiting a permanent displacement out-of-plane. On reversal of the direction of seismic displacement, the element will need to be pulled back the other way (into the building). This action will subject the diaphragm to out-of-plane floor forces, $F_{OP,i}$, which can be significant.

Consideration is required of when and where the push or pull forces develop. One side of a building has columns being pushed out of the building, while the other side is pulling the columns back in to the building.

A recommended methodology for assessing the out-of-plane forces, $F_{OP,i}$, is as follows:

- Determine the out-of-plane displacement profile for a column, etc. from the pESA.
- Using a linear elastic analysis program impose this displacement profile on the element.
- Determine the out-of-plane bending moment at the base of the element. If the displacement is sufficient to yield the base of the element then scale the moments determined by the linear elastic analysis to the overstrength of the element base.
- Determine the shear force distribution for this overstrength moment.

At each floor level, the difference in this shear force distribution is to be added to the pESA model, which is then re-run and the out-of-plane forces, $F_{OP,i}$, determined accordingly (i.e. taking the difference in out-of-plane shear in the vertical elements above and below the diaphragm being assessed).

C5E.11 Redistribution of Diaphragm Loads

It is probable that the reinforcing steel in the diaphragm may be insufficient to resist the tensions determined from the pESA.

One method to account for floor regions that may have yielding and to allow for a redistribution (plastic) of forces within the diaphragm is to adjust the section properties of the yielding members. Accordingly, adjust the stiffness of the yielding members until the yield forces are the outputs from the elastic pESA.

For each load case, it may take a couple of iterations to stabilise the redistribution of forces within the diaphragm.

For those situations when connections between the vertical lateral load resisting elements and the diaphragm are grossly overloaded (i.e. if very limited connectivity is provided) both the global building model (i.e. the analysis model used to assess the capacity of the vertical lateral load resisting elements) and the pESA analysis model may need to be adjusted so the affected vertical lateral load resisting elements are disconnected from the diaphragm.
Appendix C5F: Deformation Capacity of Precast Concrete Floor Systems

C5F.1 General

Deformation demands of the primary lateral force resisting systems can cause damage to the diaphragm structure (as a result of beam elongation or incompatible relative displacements between the floor and adjacent beams, walls or steel braced frames). Figures C5F.1 and C5F.2 illustrate two common examples of incompatible deformations between primary structure and a floor system.

Note:
The material in this section has largely been sourced from the University of Canterbury Research Report 2010-02 by Fenwick et al. (2010).

Figure C5F.1: Incompatible displacements between precast floor units and beams (Fenwick et al., 2010)
When present, precast concrete floor units effectively reinforce blocks of a diaphragm and concentrate any movement into cracks, which open up at the weak section between the floor and supporting structural elements. Where beams may form plastic hinges in a major earthquake, elongation within the plastic hinges can create wide cracks by pushing apart the beams or other structural components supporting the precast floor units. This can lead to the formation of wide cracks around the perimeter of bays of floor slabs containing prestressed precast units (refer to Figures C5F.3 and C5F.4).

Compression forces (struts) and tension forces (ties) may not be able to traverse damaged areas of floor. When assessing diaphragms, due allowance needs to be made for the loss of load paths, anticipating localised damage within the diaphragm.

Tests have shown that a wide crack does not develop where a linking slab is located between the first precast unit and a column in a perimeter frame – provided it does not have a transverse beam framing into it and the column is tied into the floor with reinforcement that can sustain the tension force given in NZS 3101:2006, clause 10.3.6 (Lindsay, 2004). Refer to Figure C5F.3(c).
C5F.2  Extent of Diaphragm Cracking

Figures C5F.3 and C5F.5(a) show the locations of wide cracks, which may limit strut and tie action in a floor. The length of these cracks round a perimeter frame (lines 1 and A in Figure C5F.5(a)) depends on the relative strength of the perimeter beams in lateral bending to the strength of reinforcement tying the floor into the beams. A method of assessing the lengths of these cracks is presented below.

Figure C5F.3: Separation crack between floor and supporting beam due to frame elongation (Fenwick et al., 2010)

Figure C5F.4: Observed separation between floor and supporting beam due to frame elongation in 2011 Canterbury earthquakes (Des Bull)
A wide crack is assumed to be one where the reinforcement tying the floor to a beam, or other structural element, has been yielded. In these zones shear transfer by conventional strut and tie type action is likely to be negligible.

The extent of cracking along an intermediate beam, such as the beam on line C in Figure C5F.5 depends on the relative magnitudes of inelastic deformation sustained in the perimeter frame (such as the frame on line 1) and an adjacent intermediate frame (such as frame on line 3 in Figure C5F.5(a)). Where the intermediate frame is flexible compared to the perimeter frame, extensive inelastic deformation together with the associated elongation may occur in the perimeter frame with no appreciable inelastic deformation in the intermediate frame.
C5F.3 Method for Assessing Crack Length

The length over which a wide crack may develop between a perimeter beam and an adjacent floor slab can be assessed from the lateral flexural strengths of the beam and the continuity reinforcement tying the floor to the beam. Figure C5F.6 shows the separation of a corner column due to elongation in beams framing into the column.

The beams are displaced laterally, opening up a wide crack at the interface between the floor slab and beam such that the strain in the reinforcement tying the beam to the floor is in excess of the yield strain. The length of the wide crack is determined by the lateral strength of the beam. If the floor slab is assumed to provide restraint to torsion the critical length, \( L_{\text{crack}} \), is given by:

\[
L_{\text{crack}} = \sqrt{\frac{2M_o}{F}} \quad \text{...C5F.1}
\]

where:

\[
M_o = \text{flexural overstrength of beam about the vertical axis}
\]

\[
F = \text{yield force of continuity reinforcing per unit length}
\]

When calculating the flexural overstrength of the beam, \( M_o \), the effects of strain hardening and axial load should be included. The axial load can be taken equal to the tension force carried by outstanding portion of the effective flange, i.e. the contribution of slab reinforcement to overstrength of plastic hinge region, as defined in NZS 3101:2006, clause 9.4.1.6.2.

Note that when the equation is applied to an intermediate column, where the precast floor units span past potential plastic hinges (such as column B on line 1 in Figure C5F.5) the axial load can be high and this can make a very considerable contribution to the flexural strength. In the calculation of \( M_o \) it should be assumed that the floor slab provides torsional restraint to the beam as this gives a conservative assessment both of the flexural strength and of the length of the wide crack.
C5F.4 Inter-storey Drift Capacity of Diaphragm Components

C5F.4.1 General

The assessment of inter-storey drift capacity of diaphragms containing precast concrete components needs to consider the following:

- loss of support of precast floor units, and
- failure of precast floor units due to seismic actions, including the consideration of incompatible displacements.

C5F.4.2 Loss of support

Overview

There are two key aspects to consider when assessing precast concrete floor units for loss of support:

- loss of support due to spalling of concrete near the front face of the support ledge and near the back of the precast floor unit, together with the movement of precast floor unit relative to the supporting beam, and
- loss of support due to failure of an unreinforced, or inadequately reinforced, supporting ledge Figure C5F.7(b). This may occur due to structural actions in the supporting elements, prying action of the precast floor unit on the support ledge, and the development of bond cracks associated with longitudinal beam reinforcing Figure C5F.8.

![Support ledge tied into beam](image1.png)
![Hollowcore supported on cover concrete](image2.png)

Figure C5F.7: Support on concrete ledge tied into the supporting element or on cover concrete (Fenwick et al., 2010)

![Bond cracks](image3.png)
![Prying action](image4.png)

Figure C5F.8: Bond cracks and tensile stresses due to prying action of precast floor units (Fenwick et al., 2010)
Loss of support does need not to be considered for a precast hollow-core floor unit if two cells at the end of the unit have been broken out and filled with reinforced concrete such that the yield force of the reinforcement exceeds twice the maximum shear force sustained by the unit. In addition, this reinforcement must be adequately anchored to sustain the yield force both in the hollow core cells and in the supporting beam.

When assessing loss of support due to spalling and relative movement the methodology in Section C2 should be followed. When assessing the adequacy of existing seating widths for loss of the support the following needs to be considered:

- inadequate allowance for construction tolerance
- movement of precast floor unit units relative to the ledge providing support due to elongation and rotation of support beams
- spalling of concrete from the front face of support ledge and back face of the precast floor unit
- creep, shrinkage and thermal movement of the floor, and
- crushing of concrete resisting the support reaction due to bearing failure.

Allowances for each of these actions are detailed below.

**Inadequate allowance for construction tolerance**

In general, precast units have been constructed on the short side to reduce problems in placing the units on supporting beams. In an assessment, ideally the construction tolerance should be measured. Where these measurements are not available it is recommended that a construction tolerance of 20 mm is assumed. This gives an initial contact length between the precast floor unit and support ledge of the dimensioned length of the support ledge minus 20 mm.

**Relative movement of floor unit due to elongation and rotation**

Elongation of plastic hinges can push beams supporting precast floor units apart and reduce the contact length between the precast units and support ledge. However, as elongation is related to the mid-depth of the beam containing the plastic hinge it is also necessary to allow for further movement between precast units and support ledge due to rotation of the supporting beam (i.e. geometric elongation) as illustrated in Figure C5F.9.

![Figure C5F.9: Displacement at support of precast unit due to elongation and rotation of support beam (Fenwick et al., 2010)](image)

Movement of support relative to precast unit equals elongation of beam plus column rotation, θ, times height between beam centre-line and support seat, h.
Displacement of structural members due to frame elongation can be calculated using the following procedure, which is based on experimental measurements. Experimental testing on structures with hollow-core floor units (Fenwick, et al., 1981; Mathews, 2004; MacPherson, 2005; Lindsay, 2004) has demonstrated that frame elongation is partially restrained by precast concrete floor units when they span parallel to the beams. Figure C5F.10 illustrates three plastic hinge elongation types.

For type U and R1 plastic hinges little restraint is provided by the floor slab and the elongation at mid-depth of the beam, $\Delta_L$, can be calculated as:

$$\Delta_L = 0.0014h_b \frac{\phi_u}{\phi_y} \leq 0.037h_b$$  \hspace{1cm} \text{...C5F.2}

where:

- $h_b$ = beam depth
- $\phi_y$ = beam first yield curvature as defined in Section C5.5.2.5
- $\phi_u$ = ultimate curvature demand on beam determined using plastic hinge lengths specified in Section C5.5.2.5.

For type R2 plastic hinges where there is a transverse beam framing into the column the elongation at mid-depth of the beam, $\Delta_L$, can be calculated in accordance with Equation C5F.3 where the terms are as defined above:

$$\Delta_L = 0.0007h_b \frac{\phi_u}{\phi_y} \leq 0.02h_b$$  \hspace{1cm} \text{...C5F.3}

Equations C5F.4 and C5F.5 are applicable to reinforced concrete beams that are sustaining inelastic deformations. Some recoverable frame elongation can still be expected at yield. Pending further study, a value in the order of 0.5% beam depth is considered appropriate for assessing the performance of nominally ductile frames.
Geometric elongation associated with movement between precast units and support ledge due to rotation of the supporting beam can be calculated as:

\[ \Delta_g = \left( \frac{h_b}{2} - h_L \right) \theta \]  

…C5F.4

where:
- \( h_b \) = beam depth
- \( h_L \) = ledge height (i.e. vertical distance between top of beam and height at which precast floor unit is supported)
- \( \theta \) = beam rotation.

Total movement of precast floor unit units relative to the ledge providing support due to elongation and rotation of support beams, \( \Delta_{rot} \), is calculated as:

\[ \Delta_{rot} = \Delta_L + \Delta_g \]  

…C5F.5

where:
- \( \Delta_L \) and \( \Delta_g \) are as defined above.

**Spalling at support**

Spalling of unarmoured concrete occurs from the front of the support ledge and the back face of the hollow core units, reducing the contact length available to support the precast units. Tests have indicated that the loss in seating length due to spalling and prying action of precast units increases with the contact length between the unit and support ledge. Assessed loss due to spalling, \( \Delta_{spall} \), is given by:

\[ \Delta_{spall} = 0.5L_s \leq 35 \text{ mm} \]  

C5F.6

where:
- \( L_s \) is the initial contact length between precast unit and support ledge.

Where a low friction bearing strip has been used the value given by Equation C5F.6 can be reduced by multiplying it by 0.75.

Spalling does not need to be consider if both the unit and the ledge are armoured.

**Creep, shrinkage and thermal actions**

Shortening of a precast floor unit due to creep, shrinkage and/or thermal strains may occur at either or both of the supports. Once a crack has been initiated at one end it is possible that all the movement in the span will occur at that end. Hence, two limiting cases should be considered: all the movement occurs at the end, or no movement occurs at the end.

Opening up a crack due to creep and shrinkage movement reduces the shear transfer that can develop across the crack. This reduces the potential prying action of the hollow core unit on the beam. In this situation the reduction in prying action can either reduce or eliminate the spalling that occurs from the back face of the hollow core unit.
Note:
In recognition of this action, the calculated movement due to creep, shrinkage and thermal strain is not added to the loss of length due to spalling. The greater loss in contact length due to spalling or to creep, shrinkage and thermal strain is assumed to apply.

For practical purposes it is recommended that the loss in support length due to creep, shrinkage and thermal strain may be taken as 0.6 mm per metre of length of the precast unit.

Bearing failure
Sufficient contact length should remain between each hollow core unit and the supporting ledge, after allowance has been made for the loss of supporting length identified above, to prevent crushing of concrete due to this reaction.

The critical reaction is likely to arise due to gravity loading plus the additional reaction induced by vertical seismic movement of the ground. The required bearing area can be calculated from the allowable bearing stress in NZS 3101: 2006, clause 16.3.

C5F.4.3 Failure of precast floor units

When assessing the capacity of a precast floor unit the following potential failure modes need to be considered:
• positive moment failure near support
• negative moment failure near support
• shear failure in negative moment zones
• incompatible displacements between precast floor units and other structural elements, and
• torsional failure of precast floor units.

Consideration of vertical seismic loading, calculated using Section 8 of NZS 1170.5:2004, should be included. Detailed guidance on how to assess the above failure modes for floors with precast concrete hollow-core units is provided in the University of Canterbury Research Report 2010-02 by Fenwick et al. (2010). Similar principles can be used to assess the performance of other types of precast concrete floor units.
Appendix C5G: Buckling of Vertical Reinforcement and Out-of-Plane Instability in Shear Walls

This appendix (a) outlines a possible approach to assessing buckling of reinforcing bars in RC elements with emphasis on shear walls and (b) provides background information on the out-of-plane instability of shear walls.

C5G.1.1 Buckling of Vertical Reinforcement

Please note that although there has been a significant amount of research into this phenomenon (Mander et al., 1984; Mau and El-Mabsout, 1989; Mau, 1990; Pantazopoulou, 1998; Rodriguez et al., 1999; Bae et al., 2005; Urmon and Mander, 2011; Rodriguez et al. 2013), guidance for assessing existing buildings is currently limited.

In particular, the effect of the cycles (reflected in the dependence of the critical strain at the onset of buckling ($\varepsilon_{s,cr}$) on the maximum tensile strain experienced by the bar before the cycle reversal takes place ($\varepsilon_{st}$) has not been incorporated in design or assessment codes or standards.

Note:

An illustration of this phenomenon and a possible definition of the buckling critical strain is shown in Figure C5G.1 with reference to a schematic strain profile in the critical section and to the stress-strain hysteresis loop of a bar located close to the extreme fibre of the wall section. Four stress-strain states (1-4) are described.

The maximum tensile strain reached in the first part of the cycle is identified as point 1 ($\varepsilon_s = \varepsilon_{st}, f_s = f_{st}$). Two strain levels are used for the same point, representing large and moderate initial elongations of the steel: $\varepsilon_{st} = 4.0\%$ and $\varepsilon_{st} = 2.5\%$, respectively. If the failure is not reached at this point and the strain reversal occurs, the steel follows the descending branch of the hysteresis loop from point 1 to the zero stresses point 2 ($\varepsilon_s = \varepsilon_0^+, f_s = 0$).

The strain associated with point 2, $\varepsilon_0^+$, can be estimated using Equation C5G.1 where $f_{st}$ is the stress in the steel at maximum elongation (point 1) and $E_s$ is the modulus of elasticity of the steel. In the non-trivial case, where the steel has entered the inelastic range in tension, $f_{st}$ can be conservatively taken as $f_y$. As a result, Equation C5G.1 becomes Equation C5G.2

\[
\varepsilon_0^+ = \varepsilon_{st} - f_{st} / E_s 
\]  

\[
\varepsilon_0^+ = \varepsilon_{st} - \varepsilon_y
\]  ...

From point 2 towards point 3, the zero strain point ($\varepsilon_s = 0$, $f_s < 0$), the bar is subjected to compression stresses, but it remains under tensile strains. If point 3 can be reached (i.e. the bar does not buckle beforehand), the bar can withstand increasing compression strains until point 4 is reached; the point where the onset of buckling occurs (Rodriguez et al., 1999, 2013). The horizontal distance between points 2 and 4, $\varepsilon_p^*$, can be calculated with Equation C5G.3 (Rodriguez et al., 2013), as a function of the restraining ratio $s_y/d_b$. If the
**Part C – Detailed Seismic Assessment**

**C5: Concrete Buildings**

**Appendix C5-57**

**Draft Version 2016_C – 10/10/2016 NZ1-9503830-62 0.62**

Critical buckling strain, \( \varepsilon_{s,cr} \), is defined with reference to the zero strain axis, it can be calculated with Equation C5G.4.

\[
\varepsilon_p^* = \frac{11-(5/4)(s_v/d_b)}{100}
\]  

**C5G.3**

\[
\varepsilon_{s,cr} = \varepsilon_{st} - \varepsilon_y - \varepsilon_p^*
\]  

**C5G.4**

Consider, as an example, the damage developed at the free end of the walls W1 and W2 presented in Figure C5G.1. These walls were part of two buildings constructed in Christchurch and were damaged during the 22 February 2011 earthquake.

In Figure C5G.1(d), it can be observed that the spacing of the confinement hoops used in W1 was large (about 300 mm), and the restraining ratio was of the order of \( s_v/d_b = 17 \), as the vertical bar had a diameter \( d_b = 18 \) mm. However, for such a large restraining ratio the formula proposed by Rodriguez et al. (2013) is no longer valid and buckling will inevitably occur before the zero strain point can be reached. That point is represented by point 5.

As shown in the same figure, in W2 the confinement hoops were spaced at a much smaller distance, preventing the vertical bar from buckling and effectively confining the concrete.

---

**Figure C5G.1: Buckling critical strain definition (Quintana-Galo et al. 2016)**
A strain limit for buckling to be used in monotonic moment-curvature analysis can be established in two ways.

- The first approach is to use the maximum strain associated to the ultimate curvature, $\varepsilon_{sm}$ to obtain $\varepsilon_{s,cr}$, such that $\varepsilon_{st} = \varepsilon_{sm}$ in Equation C5G.5. The strain $\varepsilon_{s,cr}$ should be compared with the maximum compression strain in the inverse direction of the moment. If the section is symmetric in geometry and reinforcement, $\varepsilon_{s,cr}$, can be compared with the maximum compression strain in the concrete at $\varepsilon_{cu}$ or $\varepsilon_{cu,c}$ as corresponds, which is the strain that governs in most of the cases. If the maximum strain of the steel $\varepsilon_{sm}$ controls, as occurs in members with large flanges acting in tension, there is no need to check for buckling as the reversal cannot occur. For large values of $\varepsilon_{sm}$ smaller, but close to 6%, $\varepsilon_{s,cr}$ takes positive values, indicating that buckling will occur while the bar experiences tensile strains (point 5 in Figure C5G.1).

- The second approach is to set the ultimate strain of the concrete as $\varepsilon_{cm} = \varepsilon_{cu}$ or $\varepsilon_{cu,c}$ as corresponds, and calculate the maximum tensile strain $\varepsilon_{st} = \varepsilon_{su,b}$, which is the maximum tensile strain that a bar can develop such that buckling of that bar under reversed bending actions occurs at the same time than crushing of the concrete. Setting $\varepsilon_{s,cr} = -\varepsilon_{cm}$, and $\varepsilon_{st} = \varepsilon_{su,b}$ in Equation C5G.5:

$$\varepsilon_{su,b} = -\varepsilon_{cm} + \varepsilon_{y} + \varepsilon_{p}^*$$

...C5G.5

The superscript $r$ is used to indicate that this concrete strain corresponds to the cross-section under reversed actions.

**Note:**

The ultimate curvature of structural members at buckling limit strain can be calculated using Equation X.X with $\varepsilon_{sm} = \varepsilon_{su,b}$ calculated with Equation XX.

As a general rule, if the spacing of confinement stirrups is greater than $7d_b$, as is typical of older construction practice, buckling is likely to control the capacity of the member, as the reinforcement bar after buckling does not follow a stable stress-strain path in compression (Mau, 1990).

Typical stress-strain curves for different values of $s_v/d_b$ (6.5, 10 15) are presented in Figure C5G.2 (Mau and El-Mabsout, 1989). Figure C5G.3 shows the maximum compression normalised stress and the lateral displacement of the bar for different restraining ratios.
Part C – Detailed Seismic Assessment

C5: Concrete Buildings

Appendix C5-59


Figure C5G.2: Stress-strain curves of a steel bar in compression for $s_v/d_b = 6.5, 10$ and $15$ (Mau and El-Mabsout, 1989)

Figure C5G.3: (a) normalised peak load (relative to buckling stress) and critical restraining ratio $s_v/d_b = 7$, (b) lateral displacement of the bar for different restraining ratios $s_v/d_b$ (Mau, 1990)

Note:

An indicative limit of the buckling strain limit in the steel rebars can be taken as the maximum tension strain in the steel that, given the $s/d_b$ ratio, will produce buckling at a compression strain equal to the maximum (ultimate) strain of the concrete.

As an example, for $s/d_b = 6$, $\varepsilon_y = 0.25\%$, and assuming a well confined concrete $\varepsilon_{cu} = 1\%$, the maximum strain in the steel governing the buckling would be about $3\%$.

Considering that older construction details are likely to be worse than the assumed value of $s/d_b = 6$ and $\varepsilon_{cu} = 1\%$ this would suggest that $\varepsilon_{su} = 3\%$ represents a simplistic upper limit to be adopted to account for buckling effects in section analysis. This is instead of $\varepsilon_{su} = 6\%$ assumed in Table XX considering a flexural-dominated and ideal behaviour.
Based on Equation XX, the ultimate curvature at the onset of buckling, $\phi_u^*$, and the corresponding plastic displacement, $\delta_p^*$, can be estimated using Equations C5G.6 and C5G.7 respectively.

$$\phi_u^* = \frac{\epsilon_p^*}{\gamma l_w}$$  \[C5G.6\]

$$\delta_p^* = L_p (\phi_u^* - \phi_y) (h_w - 0.5L_p)$$  \[C5G.7\]

where:

$\gamma l_w$ is shown in Figure C5G.4.

![Figure C5G.4: Definition of $\gamma l_w$ according to Rodriguez et al. (2013)](image)

### C5G.1.2 Out-of-plane Instability

Out-of-plane (or lateral) instability is currently identified as one of the common failure modes of slender rectangular RC walls. This ‘global’ mode of failure, which involves a large portion of a wall element as opposite to the ‘local’ bar buckling phenomenon where a single rebar is affected, was previously observed in experimental studies of rectangular walls. However, it was not considered as a major failure pattern until the recent earthquakes in Chile (2010) and Christchurch (2011).

**Note:**

Following the Canterbury earthquake sequence extensive numerical and experimental investigations are being carried out to scrutinise the effect of key parameters assumed to be influential in the formation of out-of-plane instability, such as residual strain and peak tensile strain at previous cycle, wall slenderness ratio, wall length, axial load ratio and cumulative inelastic cycles experienced during the earthquake.

The final aim is to develop recommendations consistent with the approach followed in this document and integrate this failure mode within the derivation of the force-displacement capacity curve of the assessed wall.

For more detailed information and preliminary results refer to Dashti et al. (2015, 2016).
Note:

Paulay and Priestley (1993) made recommendations for the prediction of the onset of out-of-plane instability based on the observed response in tests of rectangular structural walls and theoretical considerations of fundamental structural behaviour.

Because of very limited available experimental evidence, engineering judgement was relied on extensively. It was concluded that properties for inelastic buckling are more affected by wall length than by unsupported height and the major source of the instability was postulated to be the tensile strain previously experienced by the rebar rather than the maximum compression strain.

Chai and Elayer (1999) studied the out-of-plane instability of ductile RC walls by idealising the end-region of the wall as an axially loaded reinforced concrete column, as shown in Figure C5G.5. They conducted an experimental study to examine the out-of-plane instability of several reinforced concrete columns that were designed to represent the end-regions of a ductile planar reinforced concrete wall under large amplitude reversed cyclic tension and compression.

![Diagram](image)

(a) Opening of cracks under tension cycle
(b) Closing of cracks under compression cycle

Figure C5G.5: Idealisation of reinforced concrete wall in end regions: (a) opening of cracks under tension cycle; and (b) closing of cracks under compression cycle (Chai and Elayer, 1999)

Note

Based on this study, the critical influence of the maximum tensile strain on the lateral instability of slender rectangular walls was confirmed and the basic behaviour of the wall end-regions under an axial tension and compression cycle was described by axial strain versus out-of-plane displacement and axial strain versus axial force plots, as shown in Figure C5G.6. Also, based on a kinematic relation between the axial strain and the out-of-plane displacement, and the axial force versus the axial strain response, a model was developed for the prediction of the maximum tensile strain. Points (a) to (f) display different stages of the idealised column response and are briefly described in Table C5G.1.
(a) nominal axial strain versus out-of-plane displacement  
(b) nominal axial strain versus axial force

Figure C5G.6: Axial reversed cyclic response of reinforced concrete slender wall  
(Chai and Elayer, 1999)

Table C5G.1 Behaviour of wall end-region under the loading cycle shown in Figure C5G.6

<table>
<thead>
<tr>
<th>Loading</th>
<th>Unloading</th>
<th>Reloading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Path</td>
<td>o-a</td>
<td>a-b</td>
</tr>
<tr>
<td>Large tensile strain</td>
<td>Elastic strain recovery mainly in reinforcing steel</td>
<td>Reloading in compression on the cracked concrete column accompanied by an out-of-plane displacement; yielding of the reinforcement closer to the applied axial force resulting in a reduced transverse stiffness of the column and an increased out-of-plane displacement</td>
</tr>
</tbody>
</table>

As can be seen in Figure C5G.6 and Table C5G.1, the idealised column was assumed to consist of the loading stage where a large tensile strain was applied to the specimen (Path o-a), the unloading branch (Path a-b) corresponding to elastic strain recovery mainly in reinforcement steel and the reloading in compression which can be either Path b-c-d-e or Path b-c-d-f.

During Path b-c, when the axial compression is small, the compressive force in the column is resisted entirely by the reinforcement alone as the cracks are not closed, and a small out-of-plane displacement would occur due to inherent eccentricity of the axial force. The increase in axial compression would lead to yielding of the reinforcement closer to the...
applied axial force resulting in a reduced transverse stiffness of the column and an increased out-of-plane displacement.

Path c-d corresponds to compression yielding in the second layer of the reinforcement due to further increase in the axial compression which could rapidly increase the out-of-plane displacement. Response of the idealised column after Point d depends on the initial tensile strain. If the initial tensile strain is not excessive, the cracks could close at Point d resulting in decrease of out-of-plane displacement (Path d-e). The crack closure would cause significant compressive strain to develop in the compressed concrete accompanied by increase of out-of-plane displacement. In case of excessive crack opening, the following compression would not be able to close the cracks before the increase in the out-of-plane displacement results in eventual buckling of the column.