PART C

Concrete Buildings C5

The Seismic Assessment of Existing Buildings
Errata 1: Corrections to Equations in Section C5

From Monday 26 March, 2018, the following corrections shall apply to the listed equations and their definitions within Section C5 of the Engineering Assessment Guidelines (available from www.building.govt.nz)

Item:	**Equations C5.9 and C5.10 – page C5-64**
Correction:	Delete equation C5.9 and Equation C5.10
	Replacements will only be provided in the revision to the whole of Section C5.

Item:	**Equation C5.11 – page C-65**
Correction:	Replace Equation C5.11 with the following
	$\Delta_{\text{cap}} = 0.0325 L_{\text{c}} \left(1 + k_{\text{e}_{\text{bb}}} \frac{f_{\text{yt}} d_{\text{b}}}{f'_c D} \rho_{\text{st}}\right)\left(1 - \frac{N^*}{A_{\text{g}} f'_{\text{c}}}\right)\left(1 + \frac{L_{\text{c}}}{10D}\right)$ …C5.11

Item:	**Definitions following Equation C5.11 – page C-65**
Correction:	Replace the incomplete definition of ρ_{eff} with the following
	ρ_{st}= volumetric ratio of confinement reinforcement (see Table C5.6)

Item:	**Equation C5.13 – page C-66**
Correction:	Replace the definition Equation C5.13 with the following
	$\theta_y = \frac{\Delta_y}{H} = \phi_y \left(\frac{H}{3}\right)$ Effective yield rotation …C5.13

Item:	**Equation C5.37 – page C-74**
Correction:	Replace Equation C5.37 with the following pair of equations
	$V_{\text{prob,jh}} = 0.85 v_{\text{prob,jh}} b_{\text{j}} h \leq 1.92\sqrt{f'_c}\, b_j h$ …C5.37A $v_{\text{prob,jh}} = \sqrt{\left(k_{\text{j}}\sqrt{f'_{\text{c}}}\right)^2 + k_{\text{j}}\sqrt{f'_{\text{c}}}\frac{N^*}{A_{\text{g}}}}$ …C5.37B

Item:	**Equation C5.38 – page C-76**
Correction:	Replace Equation C5.38 with the following
	$V_{\text{prob,jh}} = 0.85 v_{\text{prob,jh}} b_{\text{j}} h \leq 1.92\sqrt{f'_c}\, b_j h$ …C5.38

Item:	**Equations C5.39 and C5.40 – page C-76**
Correction:	Replace Equations C5.39 and C5.40 and the text before the equation numbers with the following
	$v_{\text{prob.jh}} = \sqrt{\left(k_{\text{j}}\sqrt{f'_{\text{c}}}\right)^2 + k_{\text{j}}\sqrt{f'_{\text{c}}}(f_{\text{v}} + f_{\text{h}}) + f_{\text{v}} f_{\text{h}}}$ for tension …C5.39 $v_{\text{prob.jh}} = \sqrt{(0.6 f'_{\text{c}})^2 - 0.6 f'_{\text{c}}(f_{\text{v}} + f_{\text{h}}) + f_{\text{v}} f_{\text{h}}}$ for compression…C5.40

Item:	**Definitions following Equation C5.40 – page C-76**
Correction:	Replace the definition following Equation C5.40 with the following
	$f_{\mathrm{v}} = \frac{N^*}{A_{\mathrm{g}}}$

Document Status

Version	Date	Purpose/ Amendment Description
1	July 2017	Initial release

This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.building.govt.nz in parts:

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

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 go to www.building.govt.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.building.govt.nz

Acknowledgements

These Guidelines were prepared during the period 2014 to 2017 with extensive technical input from the following members of the Project Technical Team:

Project Technical Group Chair	
Rob Jury	Beca
Task Group Leaders	
Jitendra Bothara	Miyamoto International
Adane Gebreyohaness	Beca
Nick Harwood	Eliot Sinclair
Weng Yuen Kam	Beca
Dave McGuigan	MBIE
Stuart Oliver	Holmes Consulting Group
Stefano Pampanin	University of Canterbury

Other Contributors	
Graeme Beattie	BRANZ
Alastair Cattanach	Dunning Thornton Consultants
Phil Clayton	Beca
Charles Clifton	University of Auckland
Bruce Deam	MBIE
John Hare	Holmes Consulting Group
Jason Ingham	University of Auckland
Stuart Palmer	Tonkin & Taylor
Lou Robinson	Hadley & Robinson
Craig Stevenson	Aurecon

Project Management was provided by Deane McNulty, and editorial support provided by Ann Cunninghame and Sandy Cole.

Oversight to the development of these Guidelines was provided by a Project Steering Group comprising:

Dave Brunsdon (Chair)	Kestrel Group
Gavin Alexander	NZ Geotechnical Society
Stephen Cody	Wellington City Council
Jeff Farrell	Whakatane District Council
John Gardiner	MBIE

John Hare	SESOC
Quincy Ma, Peter Smith	NZSEE
Richard Smith	EQC
Mike Stannard	MBIE
Frances Sullivan	Local Government NZ

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 General

C5.1.1 Scope and outline of this section

This section provides guidelines for performing a Detailed Seismic Assessment (DSA) for existing reinforced concrete (RC) buildings from the material properties to section, member, element/component, sub-assembly, and ultimately the system level. Unreinforced concrete structures are not addressed.

The overall aim is to provide engineers 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 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. (For an overview of these findings refer to Pampanin, 2009, and for more details refer to Marriott, 2009; Kam, 2011; Akguzel, 2011; Genesio, 2012; 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. The effects of the 2016 Kaikoura Earthquake on taller RC buildings in Wellington, particularly those containing precast floor systems, also represent yet another opportunity to consider the actual seismic performance of this type of building.

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 of the guidelines attempts to capture these new learnings and provide up to date procedures for evaluating the vulnerability of existing RC buildings and for determining their earthquake 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 engineers with a more holistic understanding of the overall building capacity and expected performance, which is essential when determining the earthquake 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, element probable capacities and global system capacities (Sections C5.4 to C5.8), and
- brief comments on improving RC buildings (Section C5.9).

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 C5.5.4.

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

Note:

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 earthquake rating.

Sections C5.2 and 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. It is expected that an engineer, having read these sections and being familiar with them, will thereafter be able to concentrate on Sections C5.4 to C5.8 and their associated appendices, which contain the specific assessment requirements.

The appendices to this section summarise:

- the evolution of New Zealand concrete design standards and code-based reinforcing requirements (refer to Appendix C5A)
- historical concrete property requirements, design specifications and strength testing in New Zealand (Appendix C5B)
- 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 (Appendix C5C)
- material test methods for concrete and reinforcing steel (Appendix C5D), and
- the evolution of standard based design details for reinforcement and detailing (Appendix C5E).

The appendices also discuss:

- diaphragm grillage modelling (Appendix C5F)
- assessing the deformation capacity of precast concrete floor systems (Appendix C5G)
- assessing the buckling of vertical reinforcement in shear walls (Appendix C5H)
- procedure for evaluating the equivalent “moment” capacity of a joint (Appendix C5I)
- establishing the internal hierarchy of strength and sequence of mechanisms in a column (Appendix C5J).

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 is 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.

EN 1998-3:2005. *Eurocode 8: Design of structures for earthquake resistance*, *Part 3: Assessment and retrofitting of buildings*, European Committee for Standardization (CEN), Updated in 2005.

FEMA-547 (2006). *Techniques for the seismic rehabilitation of existing buildings*, Federal Emergency Management Agency, Washington, DC.

fib (2003). *Seismic assessment and retrofit of reinforced concrete buildings: State-of-the-art report*, Bulletin 24, fib Task Group 7.1, International Federation for Structural Concrete (fib), Lausanne, Switzerland.

JBDPA (2005). *Standard for seismic evaluation of existing reinforced concrete buildings, Guidelines for seismic retrofit of existing reinforced concrete buildings, and Technical manual for seismic evaluation and seismic retrofit of existing reinforced concrete buildings*, Japan Building Disaster Prevention Association, Tokyo, Japan SEE 2006, *Assessment and improvement of the structural performance of buildings in earthquakes*, New Zealand Society for Earthquake Engineering (NZSEE) Study Group, New Zealand.

NIST GCR 10-917-7, (2010). *Program plan for the development of collapse assessment and mitigation strategies for existing reinforced concrete buildings*, National Institute of Standards and Technology.

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

Pampanin, S. (2006). *Controversial aspects in seismic assessment and retrofit of structures in modern times: Understanding and implementing lessons from ancient heritage*, Bulletin of New Zealand Society for Earthquake Engineering, Vol. 39, No. 2, 120-133.

Pampanin, S. (2009). *Alternative performance-based retrofit strategies and solutions for existing R.C. buildings*, Series "Geotechnical, Geological, and Earthquake Engineering, Volume 10" Chapter 13 within the Book *Seismic risk assessment and retrofitting - with special emphasis on existing low rise structures* (Editors: Ilki, A., Karadogan, F., Pala, S. and Yuksel, E.,), Publisher Springer, 267-295.

C5.1.3 Definitions and acronyms

ADRS	Acceleration-displacement response spectrum
Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable yield capacity is exceeded. A brittle structure has little tendency to deform before it fractures.
Critical structural weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be *potential* CSWs.
Damping	The value of equivalent viscous damping corresponding to the energy dissipated by the structure, or its systems and elements, during the earthquake. It is generally used in nonlinear assessment procedures. For elastic procedures, a constant 5% damping as per NZS 1170.5:2004 is used.
Design level/ULS earthquake	Design level earthquake or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004 (refer to Section C3)
Detailed Seismic Assessment (DSA)	A seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor or ceiling or a braced roof structure) that is strongly connected to the vertical elements around it and that distributes earthquake lateral forces to vertical elements, such as walls, of the primary lateral system. Diaphragms can be classified as flexible or rigid.
Ductile/ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Elastic analysis	Structural analysis technique that relies on linear-elastic assumptions and maintains the use of linear stress-strain and force-displacement relationships. Implicit material nonlinearity (e.g. cracked section) and geometric nonlinearity may be included. Includes equivalent static analysis and modal response spectrum dynamic analysis.
Flexible diaphragm	A diaphragm which for practical purposes is considered so flexible that it is unable to transfer the earthquake loads to shear walls even if the floors/roof are well connected to the walls. Floors and roofs constructed of timber, and/or steel bracing in a URM building, or precast concrete without reinforced concrete topping fall in this category. A diaphragm with a maximum horizontal deformation along its length that is greater than or equal to twice the average inter-storey drift. In a URM building a diaphragm constructed of timber and/or steel bracing.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Nonlinear analysis	Structural analysis technique that incorporates the material nonlinearity (strength, stiffness and hysteretic behaviour) as part of the analysis. Includes nonlinear static (pushover) analysis and nonlinear time history dynamic analysis.
Non-structural item	An item within the building that is not considered to be part of either the primary or secondary structure. Non-structural items such as individual window glazing, ceilings, general building services and building contents are not typically included in the assessment of the building's earthquake rating.
OTM	Overturning moment

Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also incorporate the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Rigid diaphragm	A diaphragm that is not a flexible diaphragm
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but, nevertheless, is required to transfer inertial and vertical loads for which assessment/design by a structural engineer would be expected. Includes precast panels, curtain wall framing systems, stairs and supports to significant building services items
Serviceability limit state (SLS)	Limit state as defined in AS/NZS 1170.0:2002 (or NZS 4203:1992) being the point at which the structure can no longer be used as originally intended without repair
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (pushover) relationship for the building as a whole
Single-degree-of-freedom (SDOF)	A simple inverted pendulum system with a single mass
Structural element	Combinations of structural members that can be considered to work together; e.g. the piers and spandrels in a penetrated wall, or beams and columns in a moment resisting frame
Structural member	Individual items of a building structure, e.g. beams, columns, beam/column joints, walls, spandrels, piers
Structural sub-system	Combination of structural elements that form a recognisable means of lateral or gravity load support for a portion of the building: e.g. moment resisting frame, frame/wall. The combination of all of the sub-systems creates the structural system.
Structural system	Combinations of structural elements that form a recognisable means of lateral or gravity load support; e.g. moment resisting frame, frame/wall. Also used to describe the way in which support/restraint is provided by the foundation soils.
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS.* Note that an aspect of the building structure scoring less than 100*%NBS* but greater than or equal to 67*%NBS* is still considered to be a SW even though it is considered to represent an acceptable risk.
Ultimate limit state (seismic)	A term defined in regulations that describes the limiting capacity of a building for it to be determined to be an earthquake-prone building. This is typically taken as the probable capacity but with the additional requirement that exceeding the probable capacity must be associated with the loss of gravity support (i.e. creates a significant life safety hazard).
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings

XXX*%NBS*	The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
XXX%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

C5.1.4 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as calculated by application of these guidelines
a	Depth of the compression stress block ($=\beta c$)
A_{bb}	Displacement at the onset of bar buckling
A_{g}	Gross area of the member section
A_{r}	Wall aspect ratio
A_{s}	Area of reinforcement in tension
A_{s}'	Area of reinforcement in compression
A_{sp}	Area of spiral or circular hoop bar
A_{st}	Area of transverse reinforcement parallel to the applied shear
A_{st}	Area of transverse reinforcement parallel to the applied shear
A_{t}	Area of the transverse stirrups
A_{v}	Area of transverse shear reinforcement at spacing s
$As,_{\mathrm{eff}}$	Area of the effective steel of the slab
b_0	Effective width of the spandrel for torsion
b_{b}	Beam width
b_{c}	Column width
b_{core}	Width of column core, measured from centre to centre of the peripheral transverse reinforcement in the web
b_{eff}	Effective width of the slab
b_{j}	Effective width of the joint
b_{w}	Web width
b_{w}	Width of beam web
C	Neutral axis depth
C	Resultant of compression stresses in concrete
C'	Resultant of compression stresses in compression reinforcement
D	Section effective depth
d''	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
s	Spacing of transverse shear reinforcement
c_0	Cover to longitudinal bars
c_{u}	Neutral axis depth at ultimate curvature
d_{b}	Average diameter of longitudinal reinforcement
E_{s}	Steel elastic modulus

Symbol	Meaning
f'_{c}	Probable concrete compressive strength
f'_{cc}	Probable confined concrete compressive strength
f_{st}	Stress in the steel related to the maximum tensile strain in the first part of the cycle
f_{u}	Probable ultimate strength of the longitudinal reinforcement
f_{v}	Normal stress in the vertical direction
f_{y}	Probable yielding strength of the longitudinal reinforcement
$f_{y/slab}$	Yielding stress of the slab steel in tension
F_{yt}	Yielding stress of the transverse steel
f_{yt}	Probable yield strength of the transverse reinforcement
H	Height of the member
h_{b}	Beam height
h_{c}	Column height
h_{cr}	Vertical height of inclined crack
h_{t}	Height of the transverse beam or spandrel
h_{w}	Wall height
J_{d}	Internal couple lever arm
K	Shear stress degradation factor
K_{d}	Neutral axis depth when tension steel reaches the strain at first yield, ε_{y}
k_{j}	Coefficient for calculating the shear capacity of a joint
k_{lp}	Coefficient related to the plastic hinge calculation
k_{wall}	Shear coefficient related to concrete mechanism
l_{b}	Half of the length of the beam
L_{c}	Shear span, distance of the critical section from the point of contra flexure
l_{c}	Total length of the column
l_{cr}	Horizontal length of inclined crack
l_{d}	Theoretical development length
$l_{d,prov}$	Provided lap length
$l_{d,req}$	Required lap length
L_{p}	Plastic hinge length
L_{sp}	Strain penetration length
l_{w}	Wall length
M	Bending moment
M_{b}	Moment in the beam (at the interface with the column)

Symbol	Meaning
M_{col}	Equivalent moment in the column (at the level of the top face of the beam)
M_{f}	Residual moment capacity of an element
M_{lap}	Moment capacity of a lap splice
M_{n}	Probable flexural moment capacity of an element
$M_{\mathrm{p,wall}}$	Wall probable flexural strength
N	Axial load
N^*	Total axial load: gravity plus seismic.
$p_{\mathrm{t}}, p_{\mathrm{c}}$	Tensile and compressive average principal stresses in the joint panel
S_{n}	Nominal strength capacity
S_{o}	Overstrength capacity
S_{prob}	Probable strength capacity
s_{t}	Spacing in between stirrups in the spandrel
T	Resultant of tension stresses in tension reinforcement
V	Shear
V	Maximum nominal shear stress
V_{b}	Shear force in the beam
V_{c}	Shear resisted by the concrete mechanisms
V_{c}	Shear force in the column
v_{c}	Nominal shear stress carried by concrete mechanism
$V_{\mathrm{c,wall}}$	Shear resisted by the concrete mechanisms
v_{ch}	Nominal horizontal joint shear stress carried by a diagonal compressive strut mechanism crossing joint
V_{jh}	Average shear stress in the joint panel
V_{jh}	Horizontal joint shear force
V_{n}	Shear resisted as a result of the axial compressive load
$V_{\mathrm{n,wall}}$	Shear resisted as a result of the axial compressive load
V_{p}	Probable shear strength capacity of an element
$V_{\mathrm{p,wall}}$	Wall probable shear strength
V_{pjh}	Probable horizontal joint shear force
V_{s}	Shear resisted by the transverse shear reinforcement
$V_{\mathrm{s,wall}}$	Shear resisted by the horizontal transverse shear reinforcement
α'	Shear coefficient related to section aspect ratio
α, β	Stress block parameters
α'_{wall}	Shear coefficient related to section aspect ratio
β'	Shear coefficient related to longitudinal reinforcement ratio

Symbol	Meaning
β'_{wall}	Shear coefficient related to longitudinal reinforcement ratio
γ	Inclination angle of axial load compressive truss
$\gamma_{\mathrm{bb}} l_{\mathrm{w}}$	Wall core length
Δ_{p}	Plastic displacement
Δ_{u}	Ultimate displacement
Δ_{y}	Yielding displacement
δ^*_{p}	Plastic displacement at the onset of bar buckling
ε^+_0	Tensile strain in the steel at zero stress
$\varepsilon^{\mathrm{r}}_{\mathrm{cm}}$	Concrete strain at the onset of bar buckling (reversed actions)
$\varepsilon^*_{\mathrm{p}}$	Steel plastic strain at the onset of bar buckling
$\varepsilon_{\mathrm{cu}}$	Concrete ultimate compressive strain
ε_{s}	Tension steel strain
$\varepsilon_{\mathrm{s.cr}}$	Steel tensile strain at the onset of bar buckling (cyclic actions)
$\varepsilon_{\mathrm{sh}}$	Strain at the end of the yielding plateau
$\varepsilon_{\mathrm{st}}$	Maximum tensile strain in the steel in the first part of the cycle
$\varepsilon_{\mathrm{su,b}}$	Steel tensile strain at the onset of bar buckling (monotonic actions)
$\varepsilon_{\mathrm{su}}$	Steel ultimate tensile strain
ε_{y}	Strain at first yield of the longitudinal tension reinforcement
θ	Rotation (or drift ratio)
θ_{cr}	Average cracking angle
θ_{p}	Plastic rotation (or drift ratio)
θ_{u}	Ultimate rotation (or drift ratio)
θ_{y}	Yielding rotation (or drift ratio)
μ_{Δ}	Displacement ductility
$\mu_{\Delta\mathrm{c}}$	Displacement ductility capacity
$\mu_{\Delta\mathrm{d}}$	Displacement ductility demand
μ_{ϕ}	Curvature ductility
ρ_{eff}	Effective confinement ratio
ρ_{ℓ}	Longitudinal reinforcement ratio
ρ_{s}	Volume of transverse reinforcement to volume of concrete core ratio
ϕ	Curvature
ϕ^*_{u}	Curvature at the onset of bar buckling
ϕ_{p}	Plastic curvature

Symbol	Meaning
ϕ_u	Ultimate curvature
ϕ_y	First yield curvature
Ψ_1	Coefficient for calculating the development length
Ψ_2	Coefficient for calculating the development length
Ψ_a	Coefficient for calculating the development length
Ψ_b	Coefficient for calculating the development length

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 early 1900s. 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 engineer as it often provides valuable insight into why certain detailing decisions were made and the need to recognise the 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 (2012).

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 dramatically changed New Zealand construction practice. 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 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) and the 1939 NZS Code of Building By-Laws (NZSS 95:1939).

The 1935 by-law (NZSS 95:1935) 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 95:1935).

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 1900.8:1965; NZS 1900.9:1964) 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 (ACI 318-63, 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 (ACI 318-63, 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-resisting 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).

In 1969, J.P. Hollings (Hollings, 1969) introduced a step-by-step design procedure to achieve a beam-hinging inelastic mechanism in RC frames under seismic loading, which was a precursor of the concept of capacity design. The 1968-1970 Ministry of Work’s Code of Practice for Design of Public Buildings (Fenwick and MacRae, 2009; Megget, 2006; MOW-NZ, 1968-1970) adopted many ductile detailing recommendations from the 1966 SEAOC recommendations (SEAOC, 1966) and the 1971 ACI-318 code (ACI 318-71, 1971).

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) 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) also adopted many parts of the 1971 ACI-318 code (ACI 318-71, 1971) and some recommendations from the draft of Park and Paulay's publication (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) 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.

NZS 3101:1982 was reviewed and updated in 1995 and 2006 to reflect further knowledge from research, the revisions of the NZS 4203 loading standard (NZS 4203:1976) in 1992 and the introduction of the NZS 1170 loading standard (NZS 1170.5: 2004) in 2004.

As an example of key improvements between 1982 and 1995, both in conceptual design and required details, a potential "deficiency" 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.

Note:

The period from the late 1970s through to the 1990s is one in which 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 assumptions 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

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, non-compliance 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.

The following sections discuss the behaviour of non-ductile columns and shear walls, and also include observations made following the Canterbury earthquake sequence.

C5.3.2 Non-ductile (gravity) 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)

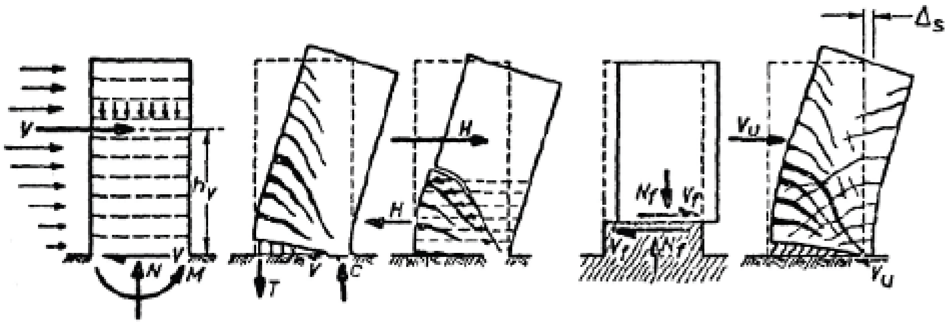
C5.3.3 Failure mechanisms for shear walls

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 C5.3 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 C5H), 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.



(a) Wall actions **(b) Flexure** **(c) Diagonal tension** **(d) Sliding shear** **(e) Hinge sliding**

Figure C5.3: Various failure modes of cantilevered shear walls (Paulay, 1981)

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 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).

In addition, a number of alternative failure mechanisms have been observed. 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 ratios.

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 ($L_{\mathrm{w}}/t_{\mathrm{w}}$)
- slenderness ratio (H/t_{w})
- longitudinal reinforcement ratio in the boundary elements and in the core (ρ_{l})
- transverse reinforcement and confinement details in the boundary regions, and
- axial load ratio ($N/f'_{\mathrm{c}}A_{\mathrm{c}}$).

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 or in the appendices) are based on the latest knowledge and will be updated as new research evidence becomes available in the near future.

C5.3.4 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:

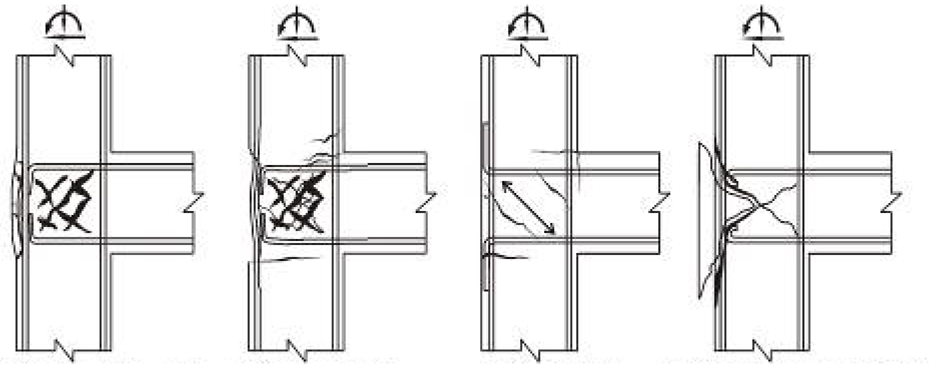
- typology (i.e. exterior or interior joints, with or without transverse beams) and
- structural details; i.e.:
  - lack or insufficient transverse reinforcement in the joint
  - type of reinforcement, i.e. plain round or deformed
  - alternative bar anchorage solutions; i.e. bent in, bent out, end-hooked,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

Figure C5.4 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 Table C5.4:

- 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.



(a) Beam bars bent in – cover cracking at back to joint **(b) Beam bars bent in – loss of joint integrity** **(c) Beam bars bent away from the joint** **(d) Plain round beam bars with end-hooks: concrete wedge mechanism**

Figure C5.4: 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), 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 C5.4(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 C5.4(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 C5.4(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.

C5.3.5 Damage observations following the Canterbury earthquakes

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 afterwards (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. Accordingly, many buildings designed and constructed prior to the 1995 standards were introduced can be expected to have inadequate levels of confinement in their columns (a 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.

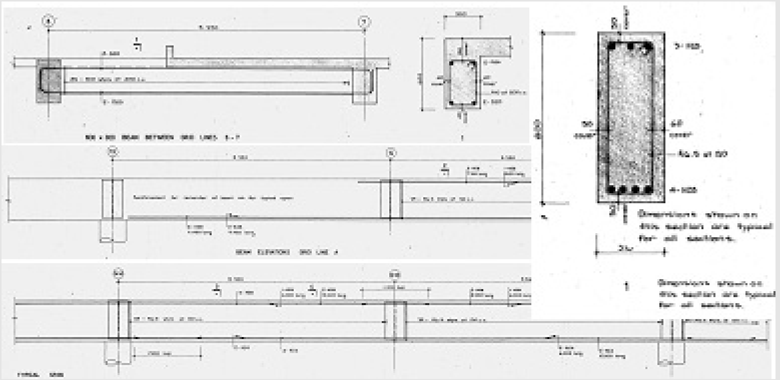
Moreover, recent focus on displacement incompatibility issues between lateral load resisting systems (i.e. walls or floors) and floor systems has shown potential SWs. Inadequate structural details could favour local damage and failure mechanisms due to beam elongation and vertical displacement incompatibilities (refer to Section C5.5.4).

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.

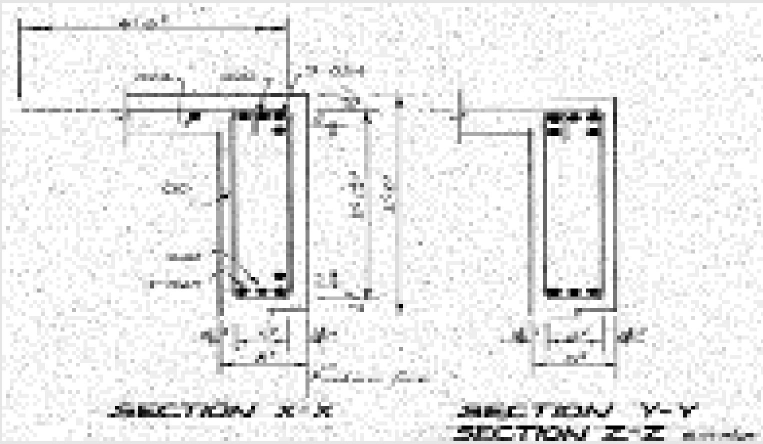
Notwithstanding these comments, modern design philosophies were also being incorporated in buildings from the late 1960s as discussed in Section C5.2.3.

Table C5.1: Typical/expected structural deficiencies and observed damage/failure mechanism in pre mid-1970s Canterbury RC buildings

<table>
<tr><th>Component or global structure</th><th>Typical deficiency</th><th>Observed damage</th></tr>
<tr><td>Beams</td><td>
Poor confinement details and transverse reinforcement in beams



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.

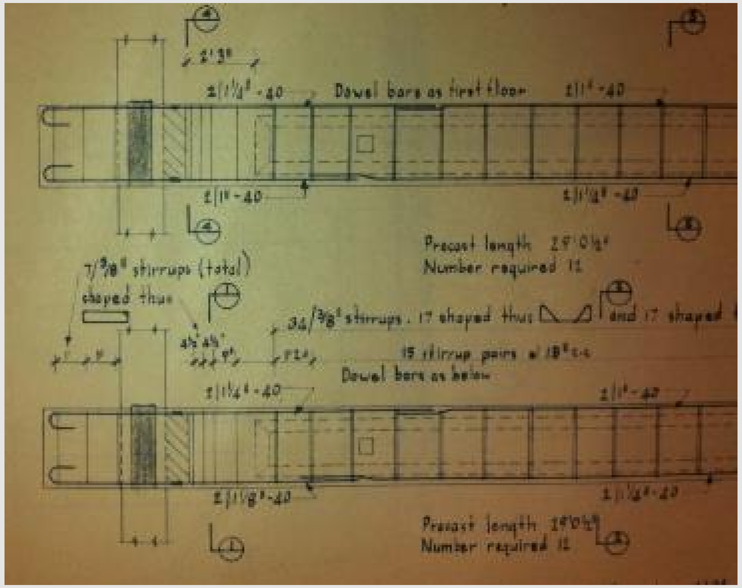


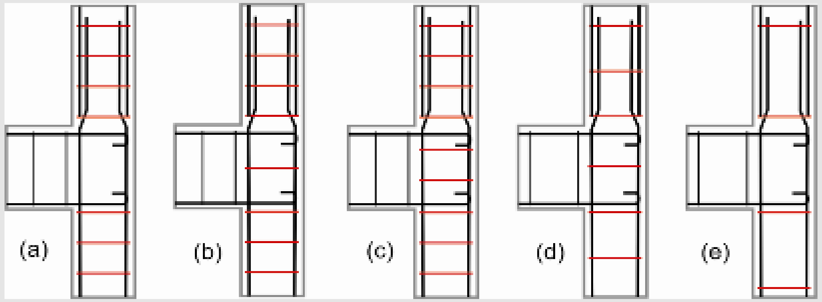
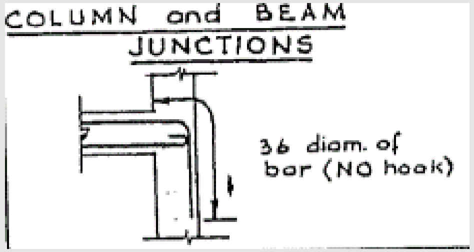
Structural drawings of beam reinforcement and confinement details
</td><td>
Flexural plastic hinge in beams, often characterised by single crack opening (refer to photo below) - especially when plain round bars adopted.

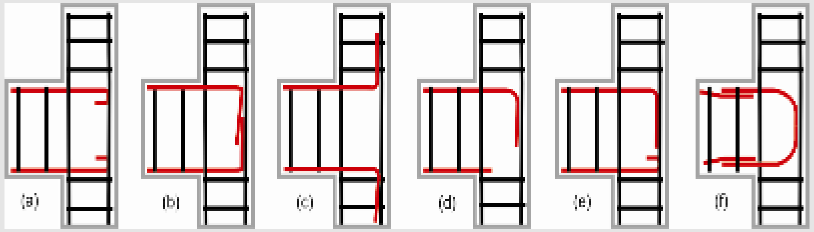
This would lead to higher deformability (fixed end rotation), lower moment capacity at a given drift demand and possibly excessive strain demand in the reinfrocing steel bars.

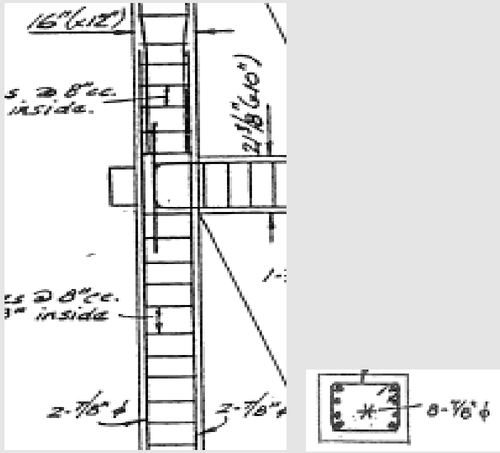
Also due to the poor confinement and transverse reinforcement details, higher level of demand could lead to premature compression-shear damage and failure in the plastic hinge region.

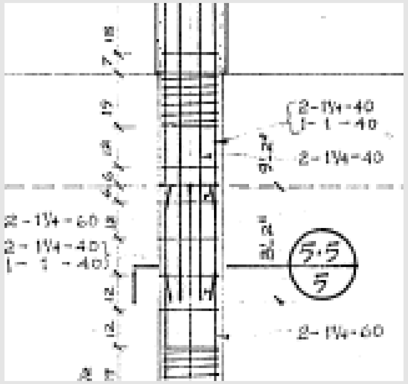
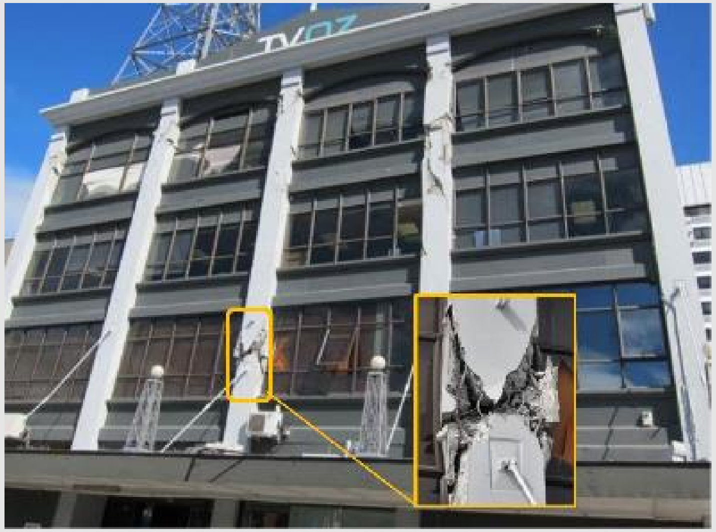

</td></tr>
</table>

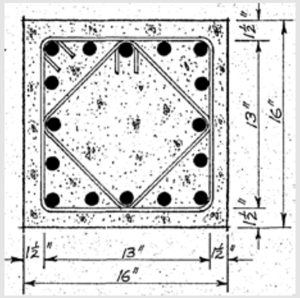
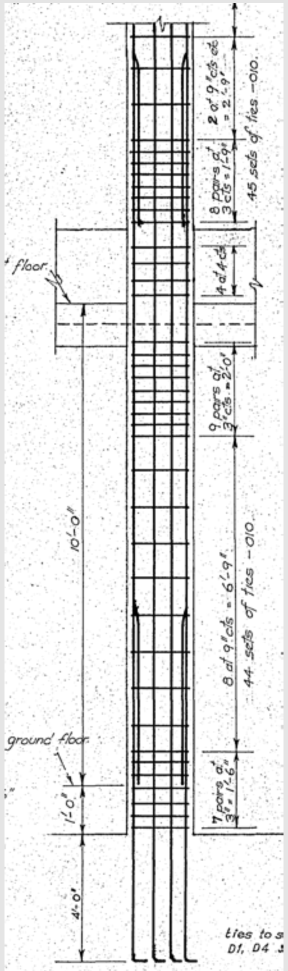
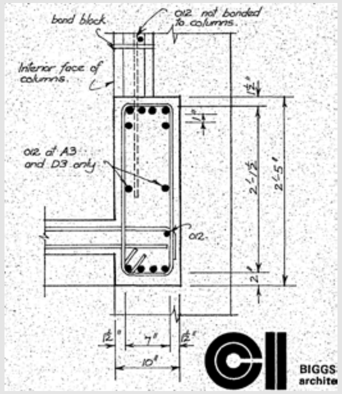
Component or global structure	Typical deficiency	Observed damage
	Inadequate anchorage of beam bars into the joint	(Refer to Joint section)
	Inadequate splice detailing (short development length, L_d, well below 40 diameters  Photo: Splices: 21” lap for D32 L_d =16 diameters); shear: 3/8” (R10) stirrups @ 18” centres (457 mm)	 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.
	Use of plain round (smooth) bars	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).

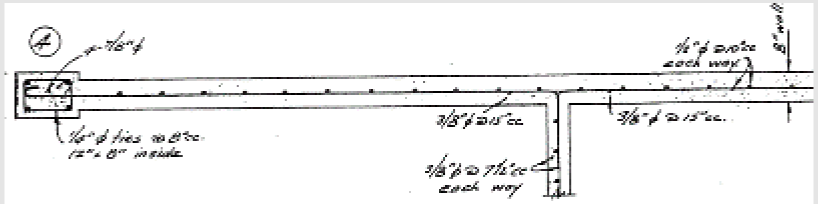
Component or global structure	Typical deficiency	Observed damage
Beam-column joints	Lack or total absence of horizontal and/or vertical transverse reinforcement in the joint panel zone.  Figure: Schematic illustrations of joint traverse reinforcement in pre-1970s buildings related to column stirrups 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	Shear damage/failure in joint area with potential loss of gravity load bearing capacity in column   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.)

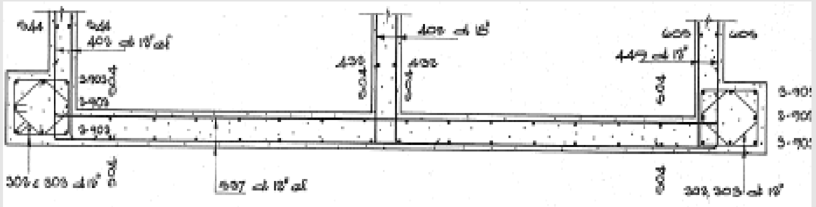
Component or global structure	Typical deficiency	Observed damage
	Inadequate anchorage of beam longitudinal bars into the joint Lack of reliable joint shear transfer mechanism beyond diagonal cracking  Figure: Alternative structural detailing of non-ductile beam-column joint: (a) 180° hooks (typical 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.	

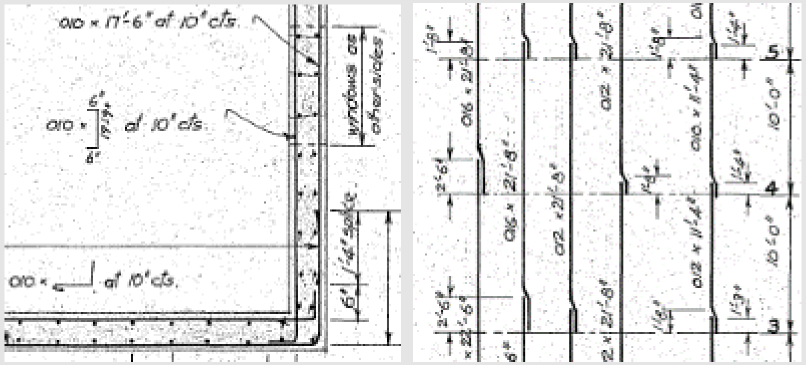
Component or global structure	Typical deficiency	Observed damage
Columns	Inadequate confinement detailing in the plastic hinge region: • not all of the bars of the longitudinal reinforcement are confined with stirrups • inadequate spacing for anti-buckling.  Figure: Structural drawings of column confinement details	Shear failure of the column at the plastic hinge Buckling of the longitudinal reinforcement at the plastic hinge  Photo: Example of shear failure and bucking of column in plastic hinge region

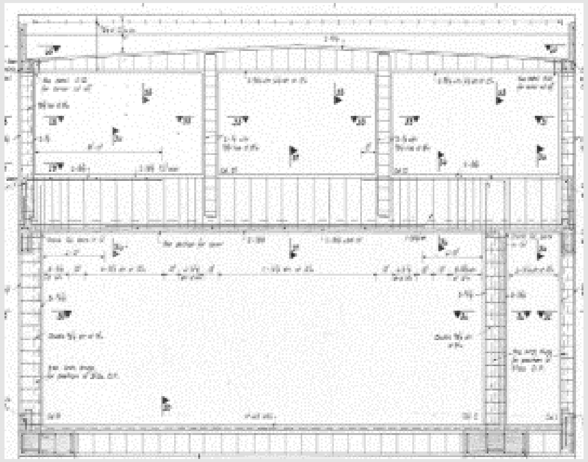
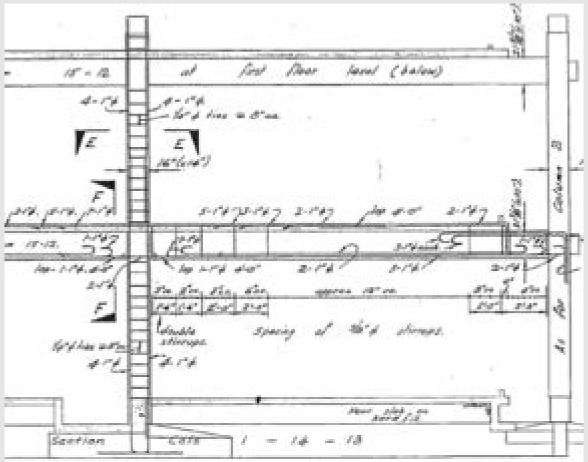
Component or global structure	Typical deficiency	Observed damage
	Inadequate lap-splice details Inadequate shear reinforcement  Figure: Structural drawing showing poor shear reinforcement details and lap splices	Potential for weak-column/strong-beam mechanism due to significant decrease in the flexural capacity of the plastic hinge Potential shear failure  Photo: Shear failure of the columns due to short-column phenomenon

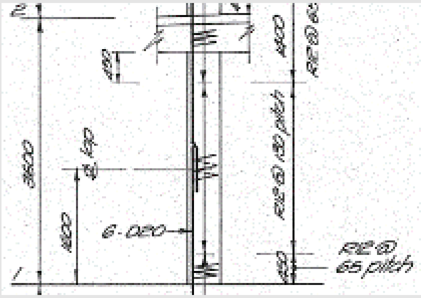
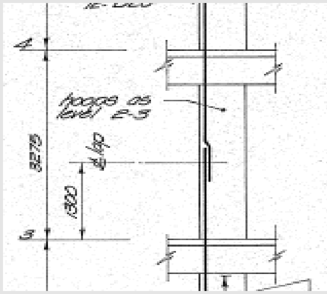
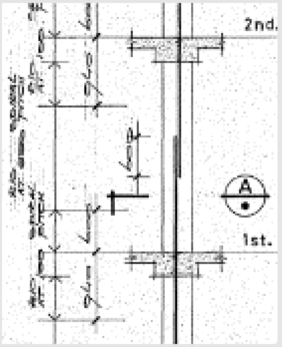
Component or global structure	Typical deficiency	Observed damage
	Short (captive) columns effects – effective shortening of the clear shear span of the columns due to presence of masonry or concrete infills, heavy spandrel beams or stiff non-structural facades   	Shear failure of columns  Photo: Short column effect and shear failure due to presence of masonry infills  Photo: Short column effect due to presence of spandrel elements (bottom)

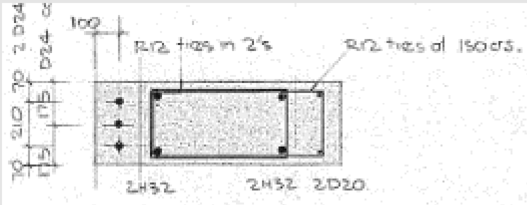
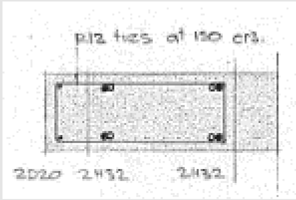
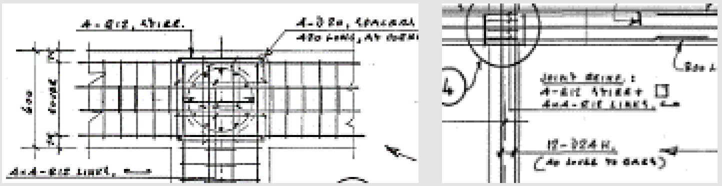
Component or global structure	Typical deficiency	Observed damage
Walls	Inadequate longitudinal reinforcement ratio  Figure: Structural drawing of a thin and singly reinforced wall	Opening of single crack in the plastic hinge region, with concentration of strain demand and potential tensile failure of longitudinal bars  Photo: Tensile failure of longitudinal rebars hidden behind a single and small (residual) crack

Component or global structure	Typical deficiency	Observed damage
	Inadequate confinement and shear reinforcement in walls  Figure: Structural drawing of confinement and shear reinforcement details in a wall	Crushing and buckling failure in the boundary regions  Photo: Wall failure due to buckled longitudinal reinforcements  Photo: Combination of buckling, single crack opening and shear sliding due to inadequate detailing

Component or global structure	Typical deficiency	Observed damage
		 Photo: Crushing of end connection in boundary regions
	Inadequate lap-splice detailing  Figure: Structural drawings of reinforcing details at lap-splices	
	Excessive wall slenderness ratio (wall height-to-thickness ratio)	Out-of-plane (lateral) instability Refer to example of associated observed damage in the following table (related to post mid-1970s walls)

Component or global structure	Typical deficiency	Observed damage
Global structure	Lack of capacity design: weak-column, strong beam mechanism, soft-storey prone   Figure: Structural drawings of weak-column, strong beam mechanisms	Severe damages to columns or joints, which can lead to global brittle failure mechanism   Photos: Severe shear damage and failure in columns

Component or global structure	Typical deficiency	Observed damage
Columns	Lap-splicing with not sufficient length and confinement. More often away from the plastic hinge region.   Figure: Structural drawings showing inadequate lap-splicing  Figure: Structural drawings showing inadequate lap-splicing	Damage due to the compromised continuity of the element, loss of moment-capacity, potential soft-storey mechanism

Component or global structure	Typical deficiency	Observed damage
	Inadequate confinement at the plastic hinge region of columns with high axial load ratio   Figure: Structural drawings of column confinement details	Shear-axial failure of columns  Photo: Compression-shear failure in columns
	Inadequate transverse reinforcement in circular columns to resist torsion  Figure: Structural drawings showing transverse reinforcement details in circular column	Torsional cracks  Photo: Torsional cracking of column

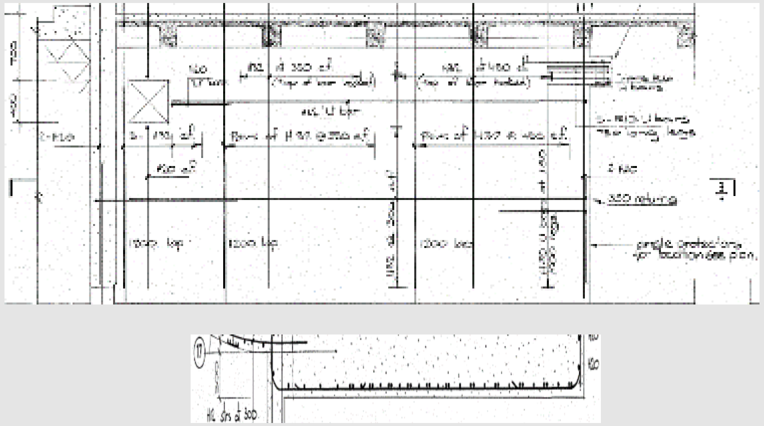
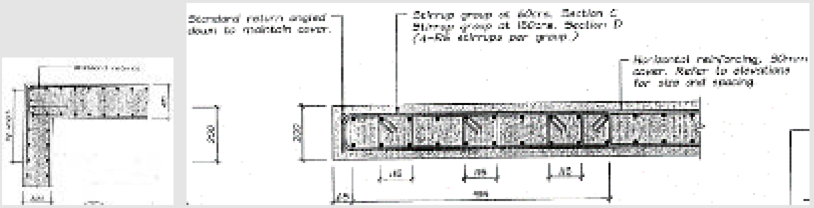
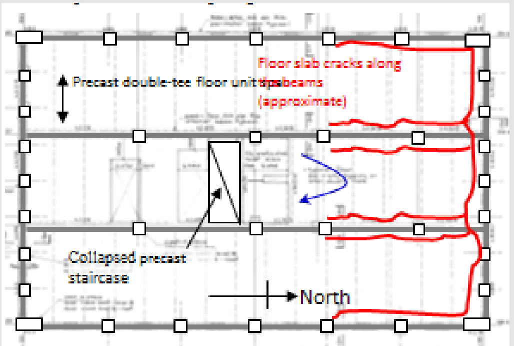
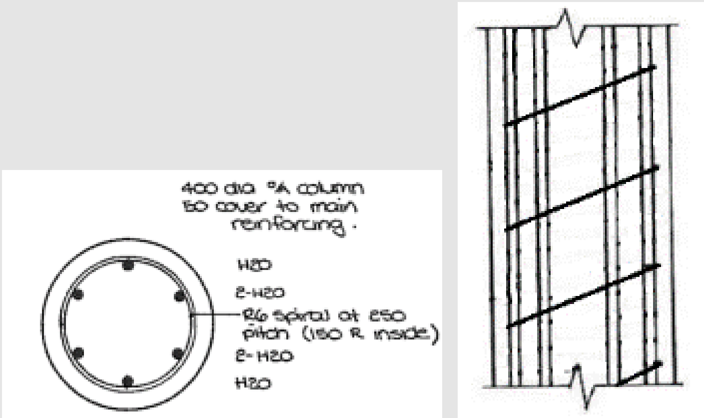
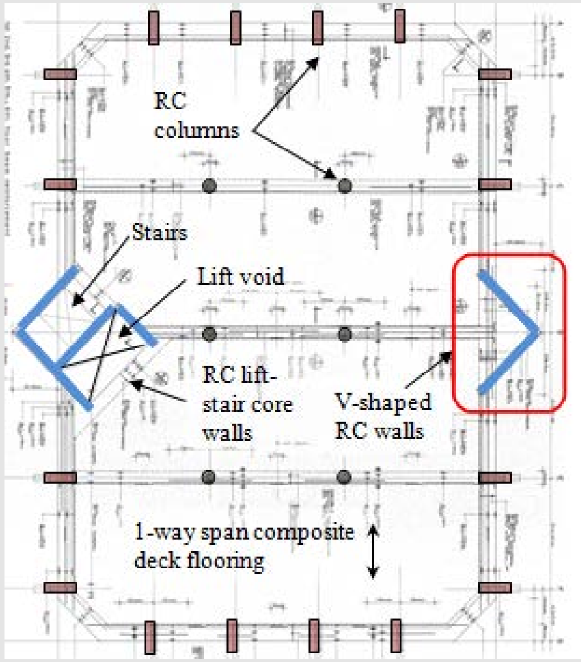
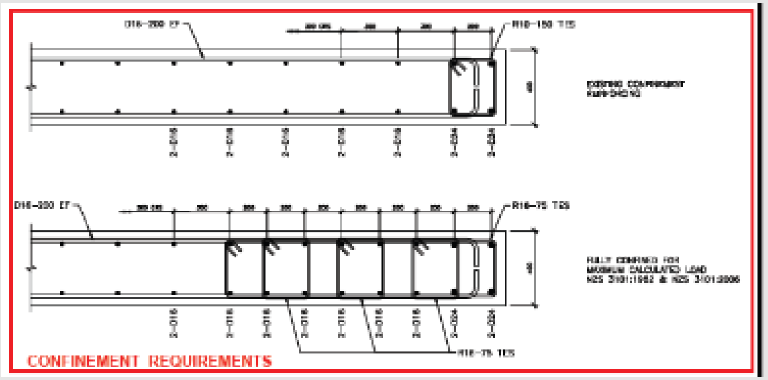
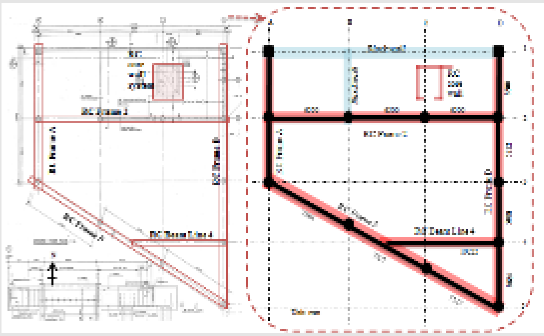
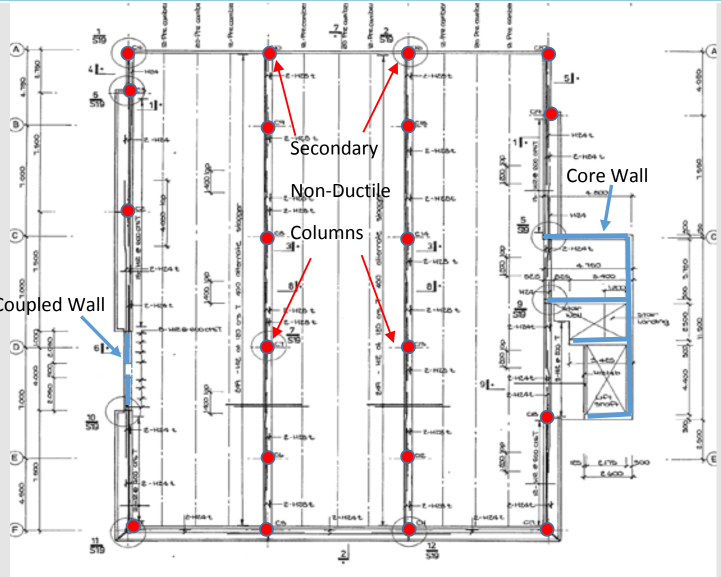
Component or global structure	Typical deficiency	Observed damage
Walls	Inadequate confinement in boundary elements as well as core area  Figure: Structural drawings of wall reinforcement and confinement details  Figure: Structural drawings of confinement details at wall corner and boundary element	Crushing, spalling of concrete; bar buckling; out-of-plane failure  Photos: Spalling of concrete at wall end, and buckling failure  Photos: Shear failure at ground floor wall

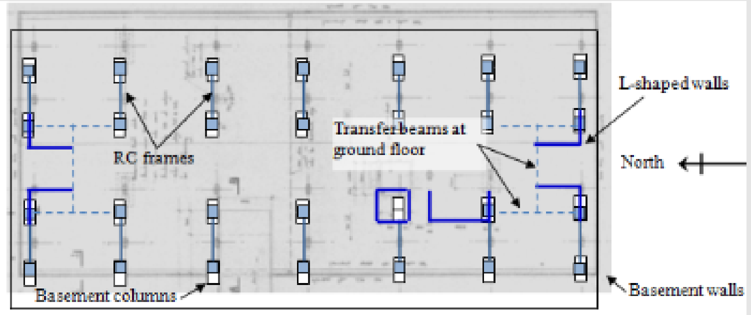
Table C5.2: Typical/expected structural deficiencies and observed damage/failure mechanism in post mid-1970s Canterbury RC buildings

Component or global structure	Typical deficiency	Observed damage
Floor/diaphragm	Beam elongation effects and lack of seating in precast floor diaphragms 	Tearing/damage to diaphragm and potential loss of seating  Photos: Damage in the diaphragm due to beam elongation; potential unseating of floor units.
Non-ductile columns	Inadequate structural detailing to provide required ductility Inadequate confinement and shear reinforcement, poor lap splices, excessive cover concrete  Photo: Example of details of pre-1995 non-ductile (secondary) columns. Large cover concrete, inadequate stirrups spacing.	Lack of capacity to sustain the imposed displacement-drift compatibly with the 3D response of the system Loss of gravity load bearing capacity at earlier level of inter-storey drift Potential catastrophic collapse of the whole building  Photos: Shear failure of pre-1995 non-ductile column details

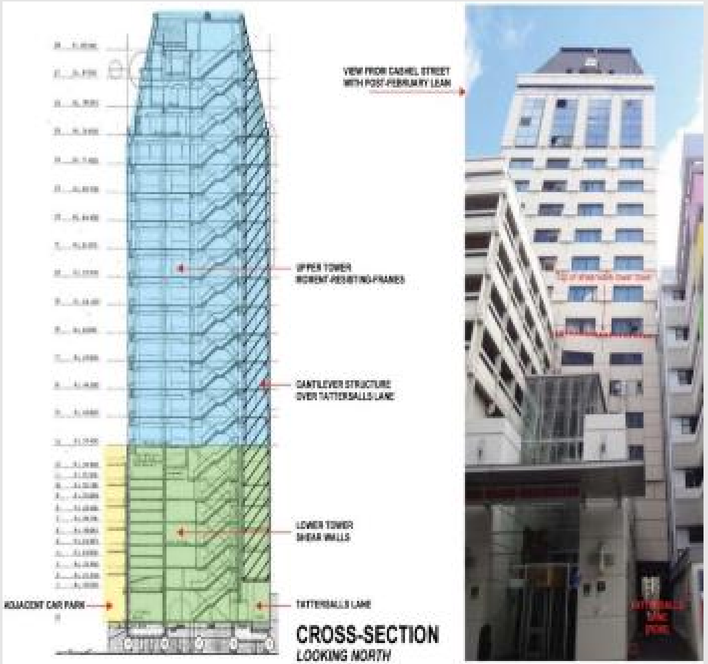
Component or global structure	Typical deficiency	Observed damage
Walls	 Flanged or irregular shaped walls Figure: Quasi-symmetric configuration of flanged-walls, yet leading to asymmetric response and inelastic torsion	Local lateral instability and concentration of damage in compression region   Photos: Crushing of well confined boundary regions and lateral instability

Component or global structure	Typical deficiency	Observed damage
	Under-designed boundary region, lack of ties in the web, inadequate design against bidirectional loading, including out-of-plane shear  Figure: Example of actual details (top) of a 1980s shear wall and equivalent redesign according to latest NZS 3101:2006 design	 Photo: Out-of-plane shear-buckling failure of shear wall
Global structure	Plan irregularity  Figure: Irregular plan	Damage due to torsional effect to components  Photos: Torsional cracks on columns

Component or global structure	Typical deficiency	Observed damage
	 Figure: Plan irregularity	 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.

Component or global structure	Typical deficiency	Observed damage
	Vertical irregularity  Figure: Schematic plan of an 11-storey building with plan and vertical irregularity	 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

Component or global structure	Typical deficiency	Observed damage
Global structure	Vertical irregularity and set backs  Photo: Vertical irregularity: set back	 Photo: Axial compression failure of ground floor column at the boundary of the setback. Transverse reinforcement: R6 spirals @ ~300-400 mm  Photo: Captive column failure at building set-back level

Component or global structure	Typical deficiency	Observed damage
Global structure	Vertical irregularity and set-backs  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.	Asymmetric behaviour leading to ratcheting response, concentration of damage in gravity load-bearing elements; e.g. base wall at the boundary with the set-back and columns under transfer 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
- knowledge of the practices of the time, and/or
- site observations of quality, and/or
- in-situ testing.

In the absence of specific information, default values for the mechanical properties of the reinforcing steel and concrete may be assumed in accordance with the relevant standards and practices at the time of construction, after first making an assessment on general material quality (particularly in relation to the concrete work). The following sections provide the intended default values.

More details on the historical material properties specifications and design requirements in New Zealand can be found in the appendices.

Note:

The extent of any in-situ testing must be based on a careful assessment of the tangible benefits that will be obtained. It will never be practical to test all materials in all locations. In-situ testing may be justifiable in situations where the critical mechanism is highly reliant on material strengths, or perhaps relative material strengths (e.g. steel grade in interconnected beams and columns) but only when judgement based on an assumed range of possible material strengths cannot indicate an appropriate outcome. “Spot” testing to ascertain the material types in generic locations might be appropriate but it is not intended that it be necessary to determine the range of properties present for a particular material.

Appendix C5D provides destructive and non-destructive techniques for gathering further information on concrete and reinforcing steel material properties if this is considered necessary.

Note:

Use of probable and overstrength member and 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 if the material strengths can be reasonably ascertained. This means tit is not intended that the engineer applies any additional factors to account for natural variation in material strengths when assessing the hierarchy within a particular mechanism.

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 determine precisely (even with extensive investigation), it is intended that a seismic assessment is based on reasonably established generic concrete properties.

C5.4.2.2 Probable compressive strength of concrete

In the absence of specific information, the probable compressive strength of concrete, $f'_{\mathrm{c,prob}}$, may be taken as the nominal 28 day compressive strength of the concrete specified for the original construction, f'_{c}, factored by 1.5 for strengths less than or equal to 40 MPa and 1.4 for strengths greater than 40 MPa.

Table C5.3 presents suggested default values for the probable compression strength of concrete when the actual specified values cannot be ascertained. These are based on typical 28 day compressive strengths specified over different time periods. If inspection indicates poor compaction, these default values may need to be reduced for column strength calculations.

Table C5.3. Default probable concrete compressive strengths

Period	Generic assumed 28 day compressive strength (MPa) f'_{c}	Default probable compressive strength (MPa) $f'_{\mathrm{c,prob}}$
1970-1981	20	30
1982-1994	25	40
1995-2005	30	45
2006-present	30	45

Note:

The actual compressive strength of old concrete is likely to exceed the specified value as a result of conservative mix design, the aging effect, and the coarser cement particles that were used. Furthermore, probable strength values should be used for assessment instead of the 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 factors of between 1.4 and 1.5 depending on the originally specified concrete strength (lower bound – fifth percentile) to obtain the probable current concrete strength is a reasonable approach. Generally accepted relationships for concrete strength gain with age indicate that enhanced strength can be expected for structures of relatively young age (beyond a year), so distinguishing for age is not considered necessary.

Recourse to default generic values is considered a reasonable approach when considered against the extent (and cost) of in-situ testing required to generate an appropriate statistical sample with no certainty of identifying areas of under-strength concrete.

C5.4.3 Reinforcing steel

C5.4.3.1 General

The mechanical properties of reinforcing steel will vary depending on the source, targeted grade and age.

The historical overview provided in Appendix C5C should provide a useful basis for selecting the expected mechanical characteristics of reinforcing steel if more specific information is not available from the building's structural and construction drawings. Whenever practicable, samples of steel from the structure should be tested to at least provide an indication of the likely grade of reinforcement that is present.

C5.4.3.2 Probable yield strength of reinforcing steel

The probable yield strength of the reinforcing steel may be taken as the mean of the upper characteristic (95^{th} percentile value) and the lower characteristic (5^{th} percentile value) yield strength.

Where the lower and upper yield strength bounds are not known, the probable yield strength of the reinforcing steel may be taken as 1.08 times the lower characteristic yield strength value.

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. The 1.08 factor is based on the lower end of this expected range.

Refer to Section C5C.1 for indicative values of the lower and upper bounds of the yield stress for specified post-1970 reinforcing steels.

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 (5^{th} percentile value) that is 15-20% greater than the specified values. Therefore, for pre-1970 reinforcing steels the probable yield strength may be taken as 1.08 x 1.15 = 1.25 times the specified yield stress values indicated in Appendix C5C.

C5.4.3.3 Probable modulus of elasticity of reinforcing steel

The probable modulus of elasticity of reinforcing steel may be taken as 200,000 MPa.

C5.4.3.4 Probable tensile capacity of reinforcing steel

The probable tensile capacity of reinforcing steel may be taken as the tensile strength given in Tables C5C.1, C5C.2 and C5C.3 appropriate for the expected age and grade of steel.

C5.4.3.5 Probable strain at tensile capacity

The probable strain in reinforcing steel at the probable tensile capacity may be taken as 0.1.

C5.4.3.6 Bar size

Typical bar sizes available before and after the mid-1970s are shown in Tables C5.4 and C5.5.

Table C5.4: Available diameters of steel reinforcement bars – before the mid-1970s

NZS 1693:1962		NZS 1879:1964		NZS 3423P:1972	
Bar designation	**d inch (mm)**	**Bar designation**	**d inch (mm)**	**Bar designation**	**d inch (mm)**
3	3/8 (9.525)	3	3/8 (9.525)		3/8 (9.525)
4	1/2 (12.7)	4	1/2 (12.7)		1/2 (12.7)
5	5/8 (15.875)	5	5/8 (15.875)		5/8 (15.875)
6	3/4 (19.05)	6	3/4 (19.05)		3/4 (19.05)
7	7/8 (22.225)	7	7/8 (22.225)		7/8 (22.225)
8	1.000 (25.4)	8	1.000 (25.4)		1.000 (25.4)
9	1 1/8 (28.575)	9	1 1/8 (28.575)		1 1/8 (28.575)
10	1 1/4 (31. 75)	10	1 1/4 (31. 75)		1 1/4 (31. 75)
11	1 3/8 (34.925)	11	1 3/8 (34.925)		1 3/8 (34.925)
12[1]	1 1/2*(38.1)	12[1]	1 1/2*(38.1)		1 1/2(38.1)
					2 (50.80)

Note:
1. Introduced in 1970

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

NZ 3402P:1973 (Stage 1)				NZ 3402P:1973 (Stage 2)			NZS 3402:1989			AS/NZS 4671:2001		
Bar designation		**d (inch)**	**d (mm)**	**Bar designation**		**d (mm)**	**Bar designation**		**d (mm)**	**Bar Designation**		**d (mm)**
R10	D10	-	10	R10	D10	10	R6	D6	6	R6	D6	6
R13	D13	½	12.7	R12	D12	12	R8	D8	8	R10	D10	10
R16	D16	-	16	R16	D16	16	R10	D10	10	R12	D12	12
R20	D20	-	20	R20	D20	20	R12	D12	12	R16	D16	16
R22	D22	7/8	22.23	R24	D24	24	R16	D16	16	R20	D20	20
R25	D25	-	25.4	R28	D28	28	R20	D20	20	R25	D25	25
R28	D28	-	28	R32	D32	32	R24	D24	24	R32	D32	32
R32	D32	-	32	R40	D40	40	R28	D28	28	R40	D40	40
R38	D38	1 ½	38.1				R32	D32	32			
							R40	D40	40			

C5.4.4 Cold-drawn welded wire mesh

The properties for cold drawn wire mesh may be taken from Appendix C5C. The maximum available strain at maximum stress should be taken as 1.5%.

C5.5 Element Probable Capacities

C5.5.1 General approach

This section sets out the procedures 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 member/element capacities is to evaluate:

- the probable flexural strength and deformation (curvature) capacity relationships for RC sections/regions and where necessary extend the relationships to rotations and inter-storey drifts
- the probable shear strength and
- the limiting effect, if any, of shear in flexural regions subjected to nonlinear deformations, reinforcing steel laps, buckling of vertical reinforcement in columns and walls, out-of-plane stability in walls, and sliding shear.

These are discussed in the sections below.

It is considered acceptable to determine probable strength capacity as the nominal strength determined from NZS 3101 using the probable material properties obtained from Section C5.4.

Note:

Member/element capacities will be dependent, in many situations, on the actions in the member/element (e.g. axial loads in columns and walls and shear in regions subjected to nonlinear deformations). Therefore, an iterative approach is likely to be employed whereby some analysis is undertaken in parallel with assessing the capacities to gain an appreciation of the likely range of actions. In this way the quantum of work required to evaluate capacities can be kept to a minimum, with a focus on only those members/elements that are likely to limit the capacity of the subsystems and systems within the building.

The probable capacity of a member/element calculated simply from consideration of section capacities may be significantly overstated if issues such as deterioration of reinforcing steel laps (particularly for round bars), buckling of poorly restrained longitudinal reinforcing steel (axially loaded members), lateral stability (thin walls) and deterioration of shear capacity in nonlinear regions are not taken into account. Guidance on how to allow for these issues is provided below.

Where specific requirements are not covered in these guidelines the probable strength capacities may be taken as the nominal capacities from NZS 3101 (i.e. ϕ =1) determined using probable material strengths. Such an approach is likely to be conservative compared with the requirements outlined below and therefore may be used in lieu of those requirements.

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 should 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 and a strength reduction factor as noted below.

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

Overstrength

The overstrength capacity 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 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 (95^{th} 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, $M_{\mathrm{o}}/M_{\mathrm{n}}$, can be taken as 1.25 (for both Grade 300 and Grade 430 steel) and 1.35 for Grade 500
- probable flexural strength, $M_{\mathrm{o}}/M_{\mathrm{prob}}$, can be taken as 1.16.

Note:

While adequate confinement can cause an increase in the concrete compressive strain and ultimate deformation capacity for columns, 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. The actual overstrength of the concrete section can be established using a moment curvature analysis, stress/strain assumptions for material strengths as noted later in this Section, and the range of expected axial loads.

Strength reduction factors

For the purposes of calculating the probable strength capacity, no strength reduction factor ϕ should be used for either flexure or shear (i.e. ϕ =1.0). Where considered necessary, a 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 can be important when assessing hierarchy of strength mechanisms for post-elastic deformation (e.g. moment resisting frames). The lower bound of flexural strength can be taken as the probable strength, and the upper bound as the overstrength.

When the hierarchy of strength mechanisms is critical to the assessment result or relied on to limit actions, the overstrength should be taken as the full overstrength at ϕ_{cap}, irrespective of the maximum curvature calculated under XXX%ULS shaking.

Note:

For lateral sway mechanisms (e.g. frame action) reliant on a hierarchy of strength it is important to also account for the variation in strength due to resulting axial loads 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).

The full overstrength should be used in assessing strength hierarchies to reflect the underlying philosophy of these guidelines that shaking is not limited to XXX%ULS shaking, and also to maintain relativity with design.

C5.5.2 Beams, columns and walls

C5.5.2.1 Flexural (moment) capacity

The probable flexural capacity at a member/element section for a beam, column or wall is represented by the generalised relationship shown in Figure C5.5. For column and wall sections the relationship shown is for a particular axial load and for all members is shown unlimited by flexural-shear action.

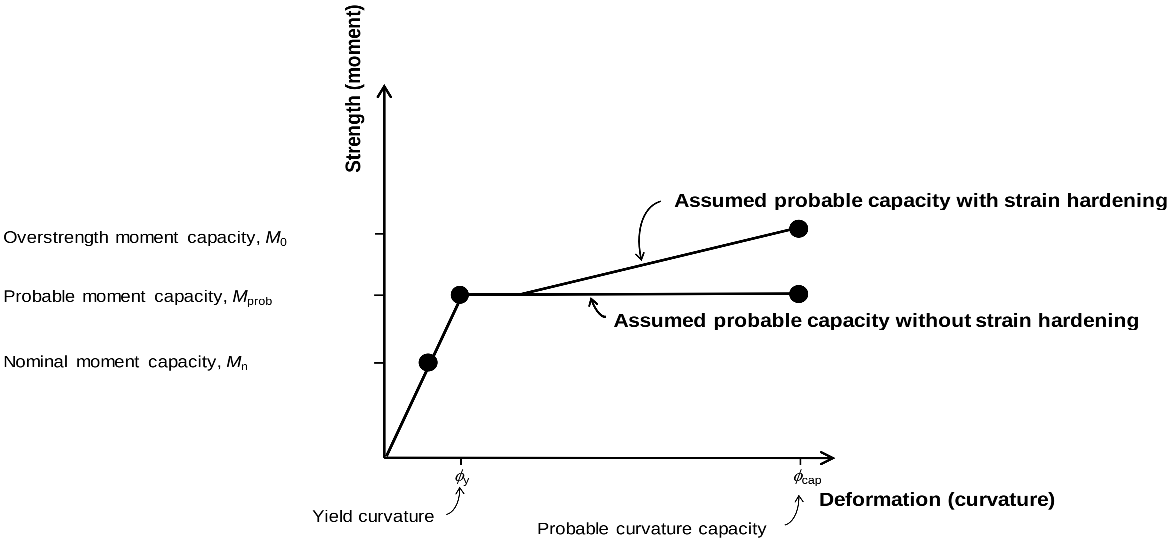


Figure C5.5: Generalised flexural strength - deformation relationship for reinforced concrete sections

The parameters that need to be determined are:

- probable flexural (bending moment) strength
- probable yield curvature
- probable curvature capacity
- overstrength capacity (when required).

The derivation of these is covered in the following sections.

Probable flexural (moment) strength

General

The probable flexural strength of member sections should be calculated using the probable material strengths determined in accordance with Section C5.4 and the standard theories for flexural strength of RC sections (Park and Paulay, 1975).

Note:

It is worth recalling that the basic theory for RC section flexural strength 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).

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/sub-assembly as discussed in subsequent sections.

The plastic hinges in 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. 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).

Based on this evidence and in order to provide a simplified procedure, the effect of bar slip on flexural strength of beams can 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.

The probable flexural strength of a 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. 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.

It has recently been shown that, depending on the structural detailing and key mechanical/geometrical parameters, an assumption of a 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 C5H. More information can be found in Dashti et al., 2015.

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, for example, to determine the likely hierarchy of strength and global mechanism, and therefore whether plastic hinging can occur in the beams or columns or both.

The axial load demands due to gravity loads and seismic actions should be accounted for when assessing the flexural strength of columns and walls.

Note:

When the axial load demands in columns and walls vary, a range will need to be considered when assessing the flexural capacity of these elements. This could have particular relevance for the development of some mechanisms dependent on a strength hierarchy.

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 C5.6). 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 the figure below. 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.

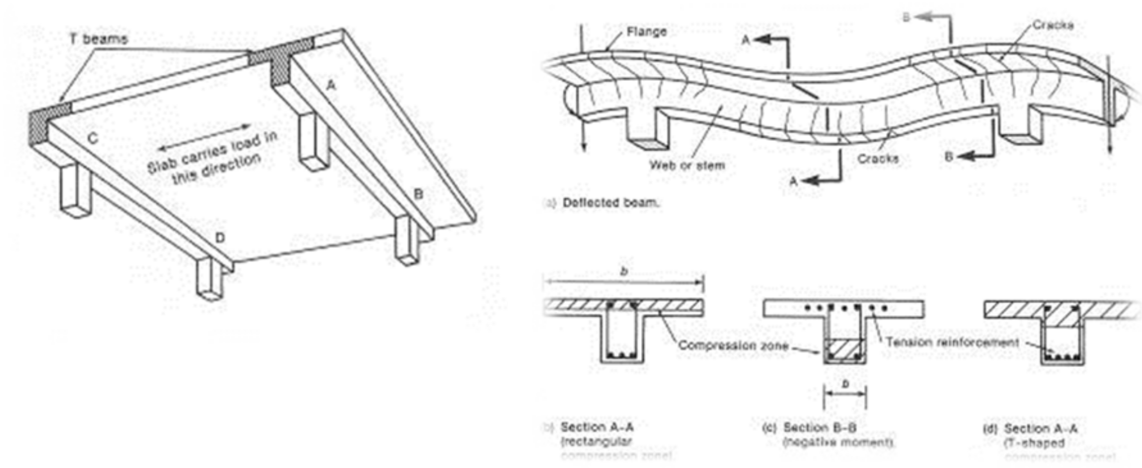


Figure C5.6: (a) Schematic monolithic one-way floor slab with beams (b) T-beam in double-bending (c) X-sections of T-beam showing different tension and compression zones (MacGregor, 1997)

Note:

The actual contributions of slab reinforcement to the negative moment flexural strength of a beam are 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 $b_{\text{eff}} = 2.2h_{\text{b}}$ (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 $b_{\text{eff}} = b_{\text{c}} + 2h_{\text{t}}$, where b_{c} is the width of the column and h_{t} is the height of the transverse beam or spandrel.

Flexural strength at lap splices

If the lap length is sufficient to develop yield (e.g. L_{ds} (approx. $20d_{\text{b}}$) for deformed bars and $2L_{\text{ds}}$ for plain bars) then the probable flexural strength capacity can be attained. For lesser lap lengths, exceedance of the capacity of the lap quickly degrades the bond strength and within one cyclic of loading the lap splice may be assumed to have failed.

When a lap splice in a beam fails in bond the contribution of those bars will need to be assumed to have been lost. However, bond failure in laps in columns and walls does not generally lead to a catastrophic failure, as the member 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 to the extent that the mechanism may also 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 against failure of a more brittle mechanism. Therefore, it is recommended to use full flexural capacity (without reduction due to lap spice failure) when assessing shear behaviour.

The probable moment capacity of a lap splice, M_{lap} , may be determined by interpolation as follows:

$$M_{\text{lap}} = M_{\text{prob}} \, {}^{L_{\text{ds,prov}}}\!/_{L_{\text{ds}}} \qquad \text{…C5.1}$$

where:

$L_{\text{ds,prov}}$ = provided lap length
L_{ds} = the development length determined from NZS 3101.

The strength of lap splices in longitudinal reinforcement in plastic hinge regions will tend to degrade during imposed cyclic loading in the post-elastic range.

In general, a structural ductility factor of greater than 2 should not be assumed possible 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 limited to 1.0 (Wallace, 1996).

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.

Note:

At a first step, and on a conservative level, the plastic rotation demand on the column, θ_{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 $l_{\text{d,req}}$ 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 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.

Probable yield curvature

The probable yield curvature, ϕ_y, can be evaluated using a section analysis but may be taken as:

$$\phi_y = \frac{\varepsilon_y}{d - kd} \qquad \text{…C5.2}$$

where:

ε_y	=	strain at the point of probable yield of the longitudinal tension reinforcement (= f_y/E_s)
d	=	effective depth of longitudinal tension reinforcement
kd	=	neutral axis depth when tension steel reaches the strain at first yield, ε_y.

In principle, and particularly for multiple layers of reinforcement in beams (and more commonly for columns), ϕ_y should be defined using a bilinear approximation (refer to Figure C5.7). The yield point so defined can be referred to as the equivalent yield point.

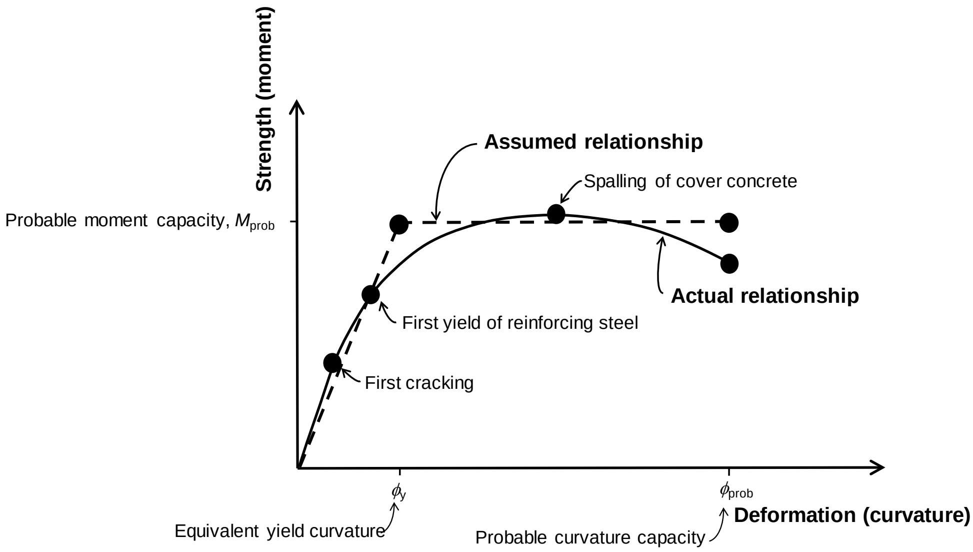


Figure C5.7: Bilinear representation of moment-curvature relationship

Priestley and Kowalsky (2000) have shown that the (equivalent) 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 and columns:

$$\phi_y = \frac{2.0\,\varepsilon_y}{h_b} \qquad \text{…C5.3}$$

For T-section beams:

$$\phi_y = \frac{1.7\varepsilon_y}{h_b} \qquad \text{…C5.4}$$

For rectangular shear walls:

$$\phi_{\mathrm{y}} = \frac{2\varepsilon_{\mathrm{y}}}{l_w} \qquad \text{...C5.5}$$

For flanged shear walls:

$$\phi_{\mathrm{y}} = \frac{1.5\varepsilon_{\mathrm{y}}}{l_w} \qquad \text{…C5.6}$$

where:

h_{b} = beam or column depth

l_{w} = wall length.

Probable curvature capacity, $\phi_{\mathbf{cap}}$

The probable curvature capacity for a beam can be taken as the lesser of:

$$\phi_{\mathrm{cap}} = \frac{\varepsilon_{\mathrm{c,max}}}{c_{\mathrm{prob}}} \qquad \text{…C5.7}$$

and:

$$\phi_{\mathrm{cap}} = \frac{\varepsilon_{\mathrm{s,max}}}{d - c_{\mathrm{prob}}} \qquad \text{…C5.8}$$

where:

c_{prob}	=	neutral axis depth at probable capacity
$\varepsilon_{\mathrm{c,max}}$	=	the accepted maximum concrete compressive strain, at the extreme fibre of the section or of the confined core region, depending on the extent of confinement of the concrete (as defined in Table C5.6 and further explained below)
$\varepsilon_{\mathrm{s,max}}$	=	the maximum accepted strain of the reinforcing steel in tension (as defined in Table C5.6)
d	=	effective depth of longitudinal tension reinforcement.

Table C5.6: Concrete and steel strain limits for calculation of probable curvature capacity

		Strain limits
Concrete	**Unconfined (including cover concrete)**	$\varepsilon_{c,max} = 0.004$
	Confined core	$$\varepsilon_{c,max} = 0.004 + \frac{1.4\rho_{st} f_{yh} \varepsilon_{ten}}{f'_{cc}} \leq 0.015$$ where ρ_{st} = volumetric ratio of confinement reinforcement $= \frac{1.5A_v}{b_c s}$ for beams and columns f_{yh} = yield strength of the confinement reinforcement ε_{ten} = available strain at the tensile strength of the reinforcing steel f'_{cc} = compression strength of the confined concrete A_v = total area of confinement reinforcement in a layer s = spacing of layers of transverse reinforcement b_c = width of core, measured from centre to centre of the peripheral transverse reinforcement
Steel		$\varepsilon_{s,max} = 0.06 \leq 0.6\, \varepsilon_{ten}$ where ε_{ten} = available strain at the tensile strength of the reinforcing steel

Note:

In general terms, for assessment purposes, the probable deformation capacity is not assumed to be limited to a value of compression strain in the extreme fibre $\varepsilon_c = 0.003$ which is the typical approach used for ULS design of new elements, but rather when either:

(i) an overall reduction in strength of more than 20% occurs, or

(ii) the confined concrete-core reaches the defined confined concrete strain limit, or

(iii) the steel reaches a much higher level of strain (e.g. $\varepsilon_s = 0.06$).

These potential deformation capabilities of an existing beam element beyond crushing and spalling of the cover concrete, $\varepsilon_c = 0.004$, can be appreciated in the moment-curvature example given in Figure C5.8 below.

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 probable curvature is strongly dependent on axial load.

This is illustrated in Figure C5.9 further below, where a poorly confined (transverse reinforcement D10@400, 2 legs only) end column of a frame with nominal axial load of $N = 0.2f'_c A_g$ is subjected to seismic axial force variations of $N_E = \pm 0.2f'_c A_g$. The yield curvatures differ by less than 10% from the mean, while the ultimate curvatures at $N = 0$ and $N = 0.4f'_c A_g$ are 61% and 263% of the value at $N = 0.2f'_c A_g$.

On the other hand, especially in columns with high axial load ratios, 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 loss of axial load capacity refer below.

In general terms, the evaluation of ultimate curvature for walls 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.

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 the probable moment capacity 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 C5H.

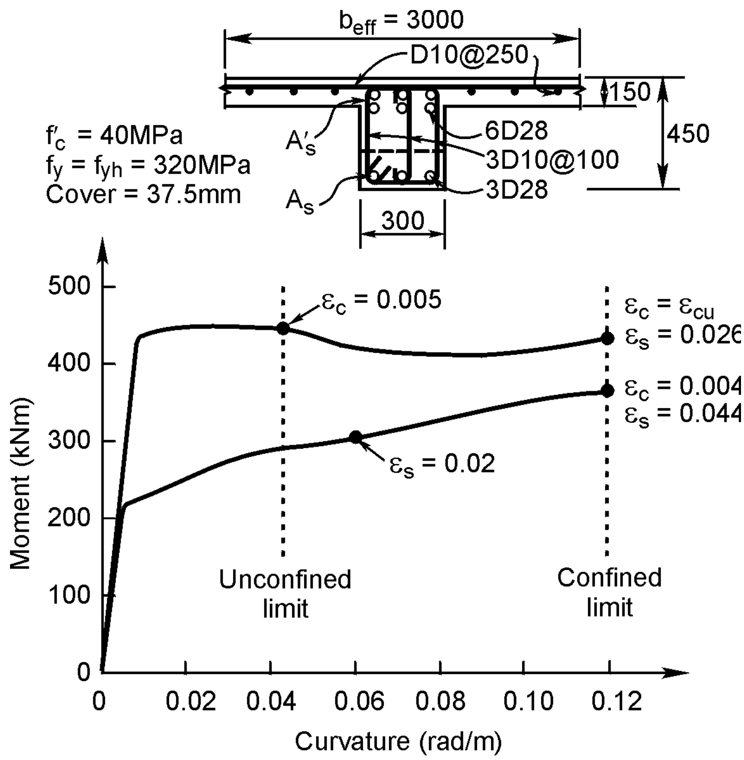


Figure C5.8: Example of a moment-curvature curve for a flanged (T or L) beam

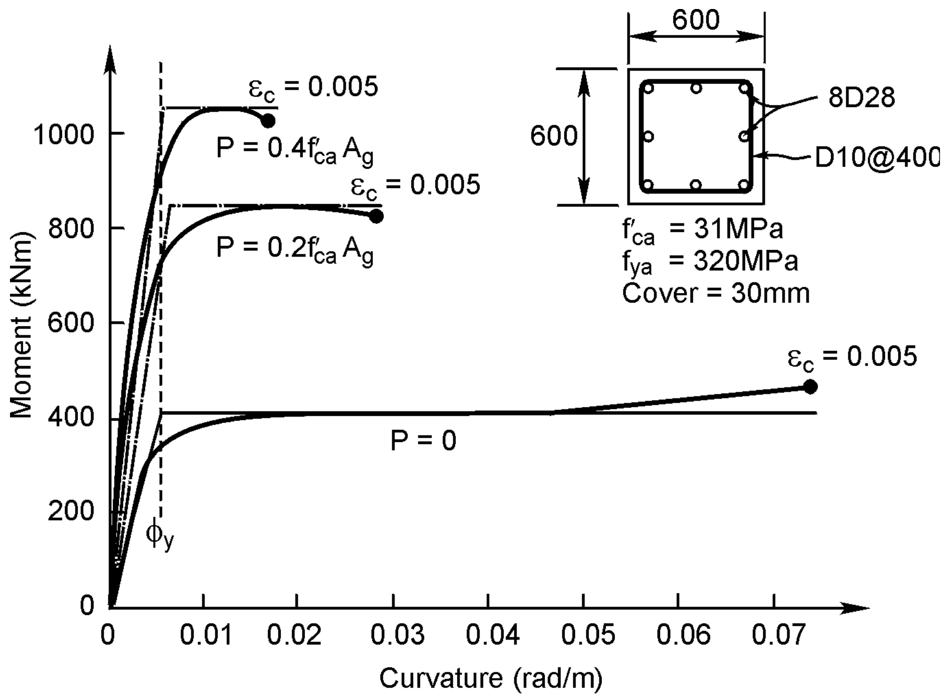


Figure C5.9: Example of a moment-curvature response of a column with poor confinement

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:

$s \geq d/2$

or

$s \geq 16d_b$

where:

d	=	effective depth of the section
d_b	=	diameter of longitudinal reinforcement

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 $f'_{cc}/f'_c = 1.0$.

A confined core may be assumed in the presence of a good level of transverse reinforcement such that:

$s < d/2$

or

$s < 12d_b$

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

Note:

The original formulation of the expression for confined concrete presented by Mander et al. (1988) can predict high levels of confined concrete strain, depending on the assumed value for the ultimate steel strain, 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 0.06 (i.e. 6%) is assumed and the values of confined concrete strain are limited to 0.015 (1.5%) in ordinary situations.

When using a moment-curvature analysis to establish, M_{prob}, ϕ_{y} and ϕ_{prob} the modelling of the materials (concrete, including the effect of confinement, and the reinforcing steel) should conform with the material properties established in Section C5.4 and the limitations set out above. To achieve consistency, modelling of the concrete should be in accordance with the modified Mander et al. expression referred to above.

Non-ductile columns

The probable capacity of non-ductile columns within the primary structure, which are described in Section C5.3.2, should be assessed in a similar manner as that recommended above for columns forming part of the lateral load resisting system.

However, given their critical role of gravity-load-carrying capacity and the lack of adequate detailing which could lead to brittle failure mechanisms, special care must 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.

Buckling of reinforcing steel in columns and walls

When the spacing of the transverse reinforcement restraining buckling of the vertical reinforcement in a wall is greater than $6d_{\mathrm{b}}$, cyclic loading and strains in the vertical reinforcing bars greater than yield are expected, the probable curvature capacity of the wall section should be limited to:

$$\phi_{\mathrm{cap}} = \frac{\varepsilon_{\mathrm{p}}^{*}}{\gamma l_{\mathrm{w}}} \qquad \text{...C5.9}$$

where:

$$\varepsilon_{\mathrm{p}}^{*} = \frac{11-(5/4)(s/d_{\mathrm{b}})}{100} \qquad \text{...C5.10}$$

γl_{w} is shown in Figure C5.10.

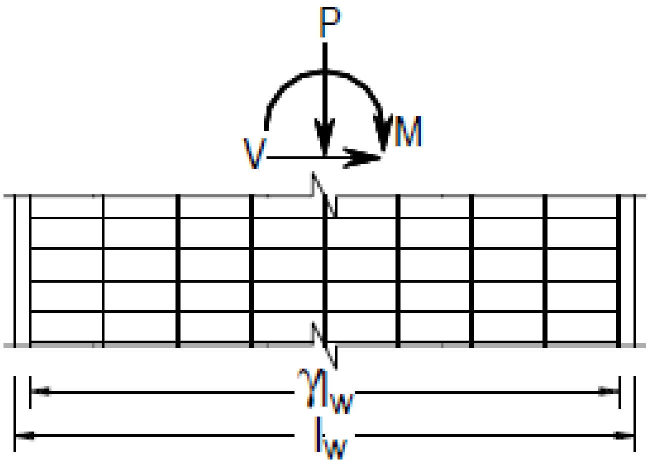


Figure C5.10: Definition of γl_w according to Rodriguez et al. (2013)

Note:

Buckling of reinforcing bars in RC elements is a complex phenomenon and, although the seismic design standards contain general detailing requirements to postpone or avoid this, there is currently limited information for assessing existing buildings. Appendix C5H discusses buckling of reinforcing bars in walls in more detail.

The following expression derived from that proposed by Berry & Eberhard (2005) can be employed to estimate the probable lateral displacement, Δ_{cap}, at which buckling of the longitudinal bars in a flexure-governed non ductile column is initiated.

$$\Delta_{cap} = 0.0325 L_c \left(1 + k_{e\_bb}\rho_{eff}\frac{d_b}{D}\right)\left(1 - \frac{N}{A_g f'_c}\right)\left(1 + \frac{L_c/2}{10D}\right) \quad \text{...C5.11}$$

where:

k_{e_bb}	=	transverse reinforcement co-oefficient
		0 for columns with $s \geq 6d_b$
		40 and 150 for rectangular columns and spiral-reinforced columns, respectively
L_c	=	distance of the critical section from the point of contraflexure
ρ_{eff}	=	effective confinement ratio
d_b	=	average diameter of longitudinal reinforcement
D	=	section effective depth

It is worth noting that the original expression proposed by Berry & Eberhard (2005) was calibrated on the drift ratio (Δ_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.

Out of plane (lateral) instability of walls

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 C5H provides an overview of the issue and a description of current knowledge on the topic.

C5.5.2.2 Evaluation of moment-rotation and force–displacement curves for members and elements

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 equivalent cantilever length and after defining a plastic hinge length. The plastic hinge length in this context is the portion of the member length over which the plastic behaviour is assumed to be concentrated and the plastic curvature is assumed to be constant.

The probable rotation capacity is defined as the sum of the yield rotation and plastic rotation capability:

$$\theta_{\mathrm{cap}} = \theta_{\mathrm{y}} + \theta_{\mathrm{p}}$$ Probable rotation capacity …C5.12

where:

$$\theta_{\mathrm{y}} = \phi_{\mathrm{y}}\left(\frac{H}{3}\right)$$ Yield rotation ...C5.13

$$\theta_{\mathrm{p}} = \left(\phi_{\mathrm{cap}} - \phi_{\mathrm{y}}\right)L_{\mathrm{p}}$$ Plastic rotation capability …C5.14

The force-displacement response can then be derived (Figure C5.11) by:

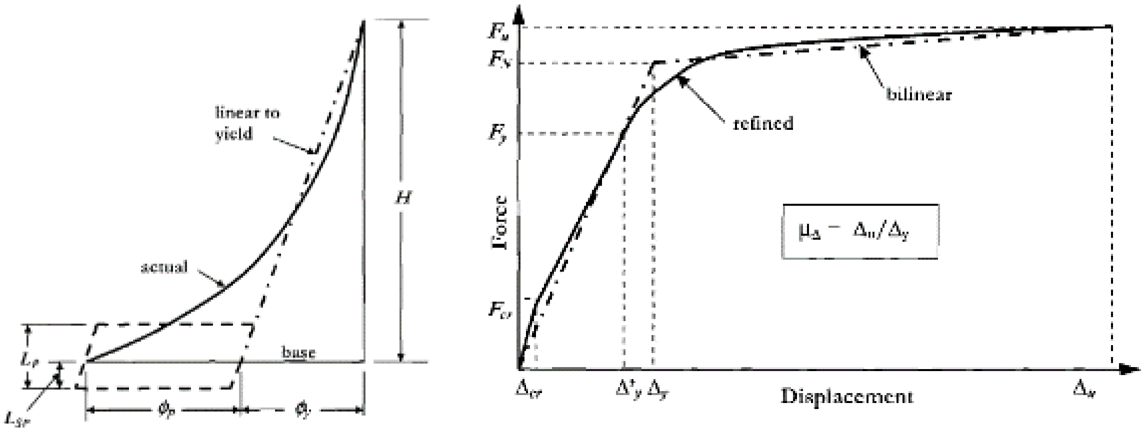
$$F = \frac{M}{H}$$ …C5.15

$$\Delta_{\mathrm{cap}} = \Delta_{\mathrm{y}} + \Delta_{\mathrm{p}}$$ Probable displacement capacity …C5.16

where:

$$\Delta_{\mathrm{y}} = \phi_{\mathrm{y}}\frac{H^2}{3}$$ Yield displacement …C5.17

$$\Delta_{\mathrm{p}} = \left(\phi_{\mathrm{cap}} - \phi_{\mathrm{y}}\right)L_{\mathrm{p}}H$$ Plastic displacement capability …C5.18



(a) **(b)**

Figure C5.11: Idealisation of: (a) curvature distribution in a cantilever scheme and (b) force-displacement curve and its bilinear approximation

Note:

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.

Plastic hinge lengths

The estimation of the plastic hinge length, L_{p}, is a delicate step in the evaluation of the probable 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 and walls.

The equivalent plastic hinge length, L_{p}, may be approximated (Priestley et al., 2007) as:

$$L_{\mathrm{p}} = kL_{\mathrm{c}} + L_{\mathrm{sp}} \qquad \text{…C5.19}$$

where:

$$k = 0.2\left(\frac{f_{\mathrm{u}}}{f_{\mathrm{y}}} - 1\right) \le 0.08 \qquad \text{…C5.20}$$

L_{c}	=	distance of the critical section from the point of contraflexure
L_{sp}	=	strain penetration = $0.022 f_{\mathrm{y}} d_{\mathrm{b}}$
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 2010-11 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
- a low longitudinal reinforcement ratio, i.e. $\rho_\ell \leq \sqrt{f'_c}/(4f_y)$ l , or
- an inadequately constructed cold joint, e.g. smooth and unroughened interfaces.

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:

$$L_P = k.L_C + 0.1l_w + L_{SP} \quad \text{...C5.21}$$

$$k = 0.2\left(\frac{f_u}{f_y} - 1\right) \leq 0.08 \quad \text{...C5.22}$$

$$L_{SP} = 0.022 f_y d_b \quad \text{...C5.23}$$

where:

L_C = distance from the critical section to the point of the contraflexure
l_w = wall length.

Note:

The values presented above for walls 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:

- a 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 $\rho_{\mathrm{n}} \geq \sqrt{f'_{\mathrm{c}}}/(4f_{\mathrm{y}})$) 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 $L_{\mathrm{p}}/5$, 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, or out-of-plane wall instability.

C5.5.2.3 Probable shear capacity

The probable shear capacity of reinforced concrete beams, columns and walls can be taken as:

$$V_{\mathrm{prob}} = 0.85(V_{\mathrm{c}}+V_{\mathrm{s}}+V_{\mathrm{n}}) \qquad \text{...C5.24}$$

where V_{c}, V_{s} and V_{n} are the shear contributions provided by the concrete mechanism, steel shear reinforcement and (where present) the axial compressive load respectively, as described below.

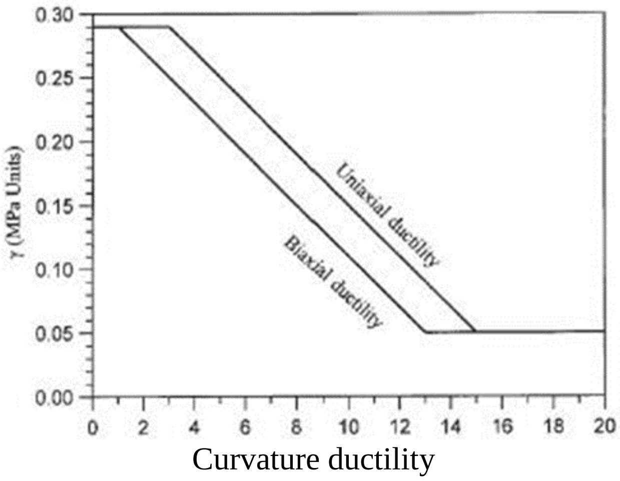
The shear contribution from the concrete, V_c, can be evaluated as:

$$V_c = \alpha\beta\gamma\sqrt{f'_c}(0.8A_g) \quad \text{…C5.25}$$

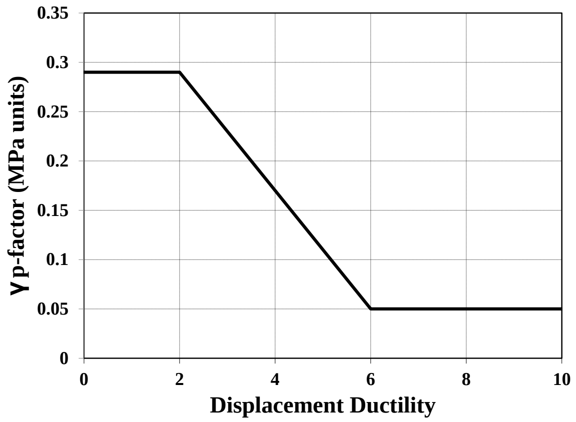
where:

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

β	=	$0.5 + 20\rho_l \leq 1$
γ	=	shear strength degradation factor (refer to Figure C5.12)
A_g	=	gross area of the member section ($b_w d$ for a beam)
b_w	=	width of section web
d	=	effective depth of section
M/V	=	ratio of moment to shear at the section
D	=	total section depth or the column diameter as appropriate
ρ_l	=	ratio of the total area of the longitudinal reinforcement.



(a) Beams and columns



(b) Walls

Figure C5.12: Concrete shear strength degradation factor as a function of ductility: curvature ductility for beams and columns and displacement ductility for walls.

The shear contribution from the shear reinforcing steel, V_s, may be evaluated as follows.

For beams assuming that the critical diagonal tension crack is inclined at 45° to the longitudinal axis of the beam:

$$V_s = \frac{A_v f_{yt} d}{s} \qquad \text{…C5.26}$$

where:

A_v	=	total effective area of hoops and cross ties in the direction of the shear force at spacing s
f_{yt}	=	probable yield strength of the transverse reinforcement
d	=	effective depth of the beam.

For columns assuming that the critical diagonal tension crack is inclined at 30° to the longitudinal axis of the column:

- For rectangular hoops:

$$V_s = \frac{A_v f_{yt} d''}{s} \cot 30^{\circ} \qquad \text{…C5.27}$$

- For spirals or circular hoops:

$$V_s = \frac{\pi}{2} \frac{A_{sp} f_{yt} d''}{s} \cot 30^{\circ} \qquad \text{…C5.28}$$

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''	=	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.

For walls the shear contribution of the effective horizontal reinforcing steel, V_s, may be evaluated as follows:

$$V_s = \frac{A_v f_{yh} h_{cr}}{\mathrm{s}} \qquad \text{…C5.29}$$

where:

$$h_{cr} = \frac{l'}{\tan \theta_{cr}} \leq h_w \qquad \text{…C5.30}$$

$$l' = l_w - c - c_0 \qquad \text{…C5.31}$$

$$\theta_{cr} = 45 - 7.5 \left(\frac{M}{V.l_w}\right) \geq 30^{\circ} \qquad \text{…C5.32}$$

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.

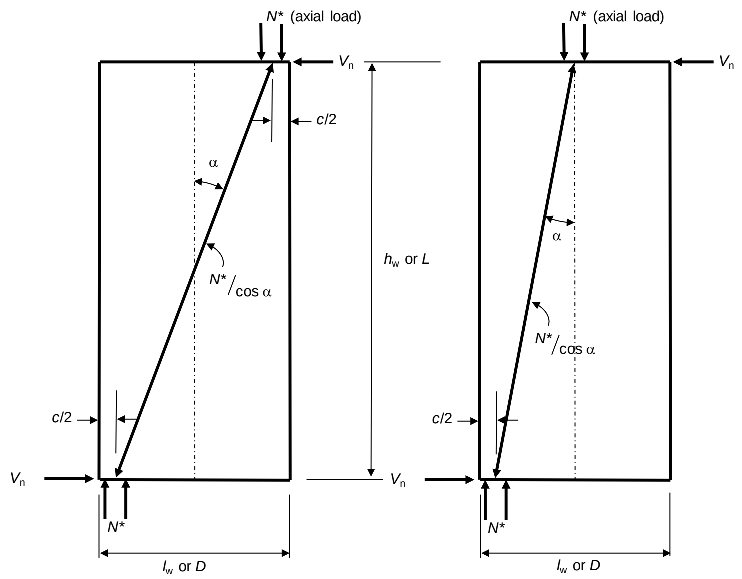
The shear resisted as a result of the axial compressive load N^* is given by:

$$V_{\mathrm{n}} = N^* \tan\alpha \qquad \ldots\text{C5.33}$$

where:

α = for a column or wall in double curvature (reverse bending), the angle between the longitudinal axis of the member and the straight line between the centroids of the concrete compressive forces of the member section at the top and bottom of the column (refer to Figure C5.13(a)).
for a cantilever column or wall (single bending), the angle between the longitudinal axis of the member and the straight line between the centroid of the member section at the top and the centroid of the concrete compression force of the member section at the base (refer to Figure C5.13(b))

N = axial compressive load.



(a) Reverse Bending **(b) Single Bending**

Figure C5.13: Contribution of compressive axial force to shear strength in a column or wall based on Priestley et al. (1994, 1995, 2007)

Therefore, for a cantilever:

$$V_{\mathrm{n}} = N^{*}\left(\frac{l_{\mathrm{w}}-c}{2h_{\mathrm{w}}}\right) = N^{*}\left(\frac{D-c}{2L}\right) \qquad \text{…C5.34}$$

or for a member with reverse bending:

$$V_{\mathrm{n}} = N^{*}\left(\frac{l_{\mathrm{w}}-c}{h_{\mathrm{w}}}\right) = N^{*}\left(\frac{D-c}{L}\right) \qquad \text{…C5.35}$$

where:

h_{w}	=	wall height
L	=	column height
D	=	column depth
c	=	the depth of the compression zone
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) for the evaluation of the shear capacity of columns.

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, Δ_{s}, can be roughly estimated from (Elwood and Moehle, 2005). In the context of these guidelines, Δ_{s}, is to be considered as the probable drift/displacement based limit associated with the evaluation of *%NBS*:

$$\Delta_{\mathrm{s}=}L_{\mathrm{c}}\left(0.03 + 4\rho_{\mathrm{s}} - 0.024\frac{v}{\sqrt{f'_{\mathrm{c}}}} - 0.025\frac{P}{A_{\mathrm{g}}f'_{\mathrm{c}}}\right) \geq 0.01L_{\mathrm{c}} \qquad \text{…C5.36}$$

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 significant critical structural weaknesses (CSWs).

C5.5.3 Beam-column joints

C5.5.3.1 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:

$$V_{\mathrm{prob,jh}} = 0.85 v_{\mathrm{prob,ch}} b_{\mathrm{j}} h$$

$$= 0.85 k_{\mathrm{j}} \sqrt{f'_{\mathrm{c}}} \sqrt{\left(1 + \frac{N^*}{A_{\mathrm{g}} k \sqrt{f'_{\mathrm{c}}}}\right)} b_{\mathrm{j}} h \le 1.92 \sqrt{f'_{\mathrm{c}}} b_{\mathrm{j}} h \qquad \text{…C5.37}$$

where:

$v_{\mathrm{prob,ch}}$	=	probable horizontal joint shear stress capacity of the 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
k_{j}	=	Coefficient for calculating the shear capacity of a joint

The following values for k_{j} should be used:

- for interior joints, k_{j} = 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_{j} = 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_{j} = 0.3
- for exterior joints with beam longitudinal (plain round) bars anchored with end hooks, k_{j} = 0.2.

Note:

These recommended values for k_{j} 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, $\sqrt{\left(1+\frac{N^*}{A_g k \sqrt{f'_c}}\right)}$ was obtained by assuming that the diagonal (principal) tensile strength, p_t, of the concrete was $p_t = k_j\sqrt{f'_c}$ 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).

The factor of 0.85 has been included in Equation C5.37 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 probable shear stress $v_{prob,jh}$, 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 C5.14 shows strength degradation curves p_t versus γ (shear deformation) as well as p_t versus drift presented in literature and based on extensive experimental tests.

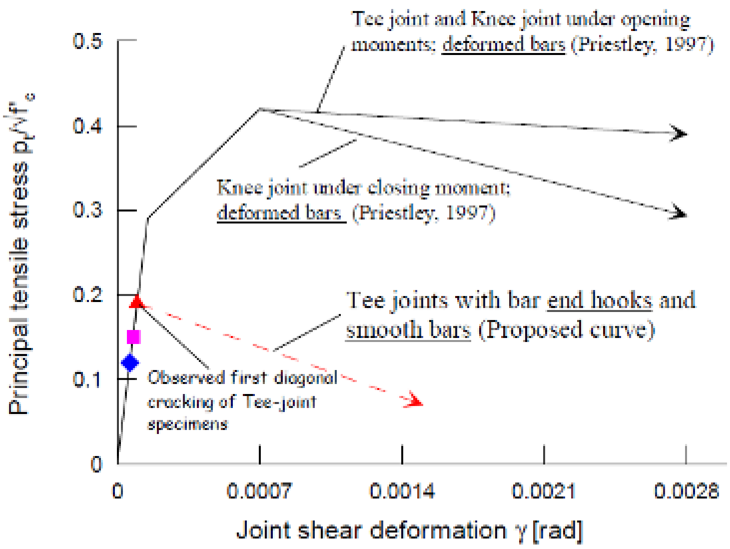


Figure C5.14: Strength degradation curves for exterior joints (Pampanin et al., 2002)

Indicative deformation limits at ULS for exterior beam column joints with no shear reinforcement, expressed in terms of shear deformation, γ [rad], and inter-storey drift, θ, are reported in Table C5.7.

In the case of interior joints, given the possibility to develop a joint shear transfer mechanism via diagonal compression strut the deformation limits of Table C5.7 can be increased by approximately 50%.

Table C5.7: Suggested ULS deformation limits for exterior joints with no shear reinforcement (modified after Magenes and Pampanin, 2004)

	ULS deformation limits
Shear deformation, γ [rad]	$0.01 \leq \gamma < 0.015$
Drift, θ [%]	$1.2\% \leq \theta < 1.8\%$

Note:

The deformation limits proposed above are based on experimental and numerical investigations on beam-column joint subassemblies and frame systems.

It is worth noting that the inter-storey 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:

$$V_{\mathrm{prob,jh}} = 0.85 v_{\mathrm{prob,jh}} b_{\mathrm{j}} h \qquad \text{...C5.38}$$

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

$$v_{\mathrm{prob.jh}} = 0.85\, k_{\mathrm{j}} \sqrt{f'_{\mathrm{c}}} \sqrt{1 + k\sqrt{f'_{\mathrm{c}}}(f_{\mathrm{v}} + f_{\mathrm{h}}) + f_{\mathrm{v}} f_{\mathrm{h}}} \quad \text{for exterior joints} \quad \text{...C5.39}$$

$$v_{\mathrm{prob,jh}} = 0.85\, k_{\mathrm{j}} f'_{\mathrm{c}} \sqrt{1 + k f'_{\mathrm{c}}(f_{\mathrm{v}} + f_{\mathrm{h}}) + f_{\mathrm{v}} f_{\mathrm{h}}} \quad \text{for interior joints} \quad \text{...C5.40}$$

where:

k_{j} = Coefficient for calculating the shear capacity of a joint

f_{v} = $\frac{N}{A_{\mathrm{g}}}$ is the axial load stress on the joint

f_{h} = $\frac{A_{\mathrm{st}} f_{\mathrm{sy}}}{b_{\mathrm{j}} h_{\mathrm{b}}}$ 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 $f_{\mathrm{h}} = 0$ (and after substituting the definition of principal tensile stress, p_{t}, as a function of nominal shear stress, $v_{\mathrm{prob,jh}}$ and the axial load stress f_{v}) the general equation for joints with shear reinforcement (Equation C5.38) converges to the equation for joints with no shear reinforcement (Equation C5.36).

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 $p_{\mathrm{t}} = k\sqrt{f'_{\mathrm{c}}}$) can be used for exterior joints.

For interior joints a similar expression based on principal compression stresses is obtained by replacing $p_{\mathrm{t}} = k\sqrt{f'_{\mathrm{c}}}$ with $p_{\mathrm{c}} = k_{\mathrm{c}} f'_{\mathrm{c}}$ and assuming $k = 0.6$ for critical damage level.

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 sub-assembly – 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 C5.15).

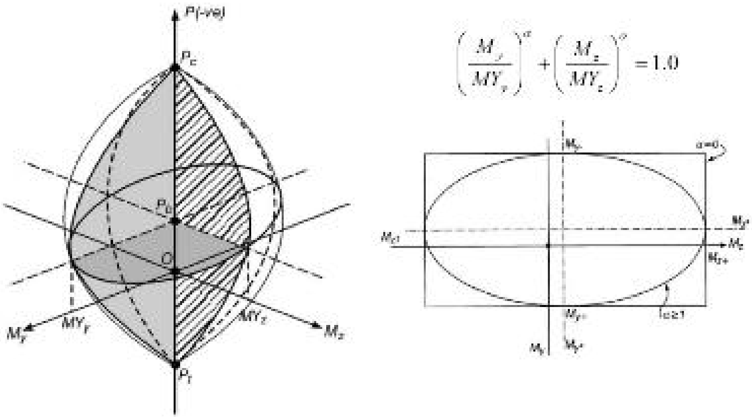


Figure C5.15: Conceptual moment-axial load ($M_{\mathrm{y}} - M_{\mathrm{z}} - P$) or shear-axial load ($V_{\mathrm{y}} - V_{\mathrm{z}} - P$) interaction surface for a reinforced concrete element (including beam-column joint) subjected to bi-axial loading

In the absence of more detailed study or evidence, a reduction of 30% on the probable joint shear (strength) capacity within the sub-assembly hierarchy of strength may be made when the joint is subjected to bidirectional loading. Also, it is suggested that the lower bounds of the deformation limit states indicated in Table C5.7 are adopted to account for the effect of bidirectional loading.

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.

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).

C5.5.4 Concrete floor diaphragms

C5.5.4.1 General

For most concrete diaphragms the in-plane deformations associated with diaphragm actions will be negligible. Therefore, the assumption of rigid diaphragm behaviour is likely to be generally 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) to avoid 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.

C5.5.4.2 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 C5.16).

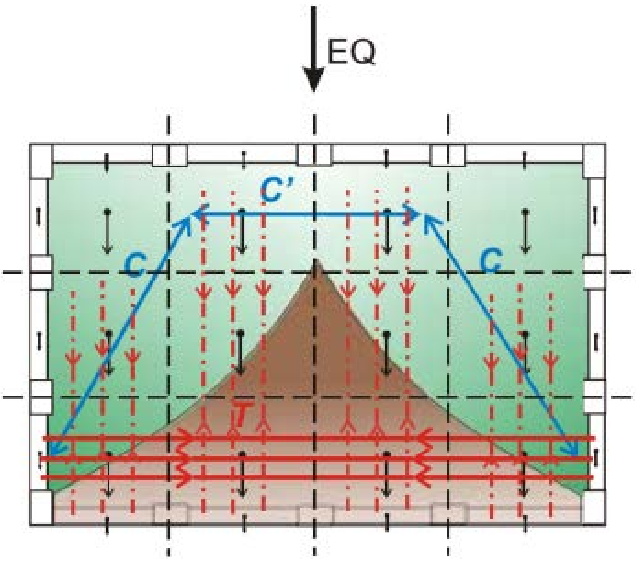


Figure C5.16: Example of a hand-drawn strut and tie solution for simple building (Holmes, 2015)

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 C5.17and C5.18. Further details of the diaphragm grillage modelling methodology are provided in Appendix C5F.

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 C5.5.4.3 below).

Step 3

Calculate probable capacities of diaphragm collector, tie and strut elements using available structural drawings and site investigation data (refer Section C5.5.4.4).

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, ϕ_{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”, 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.

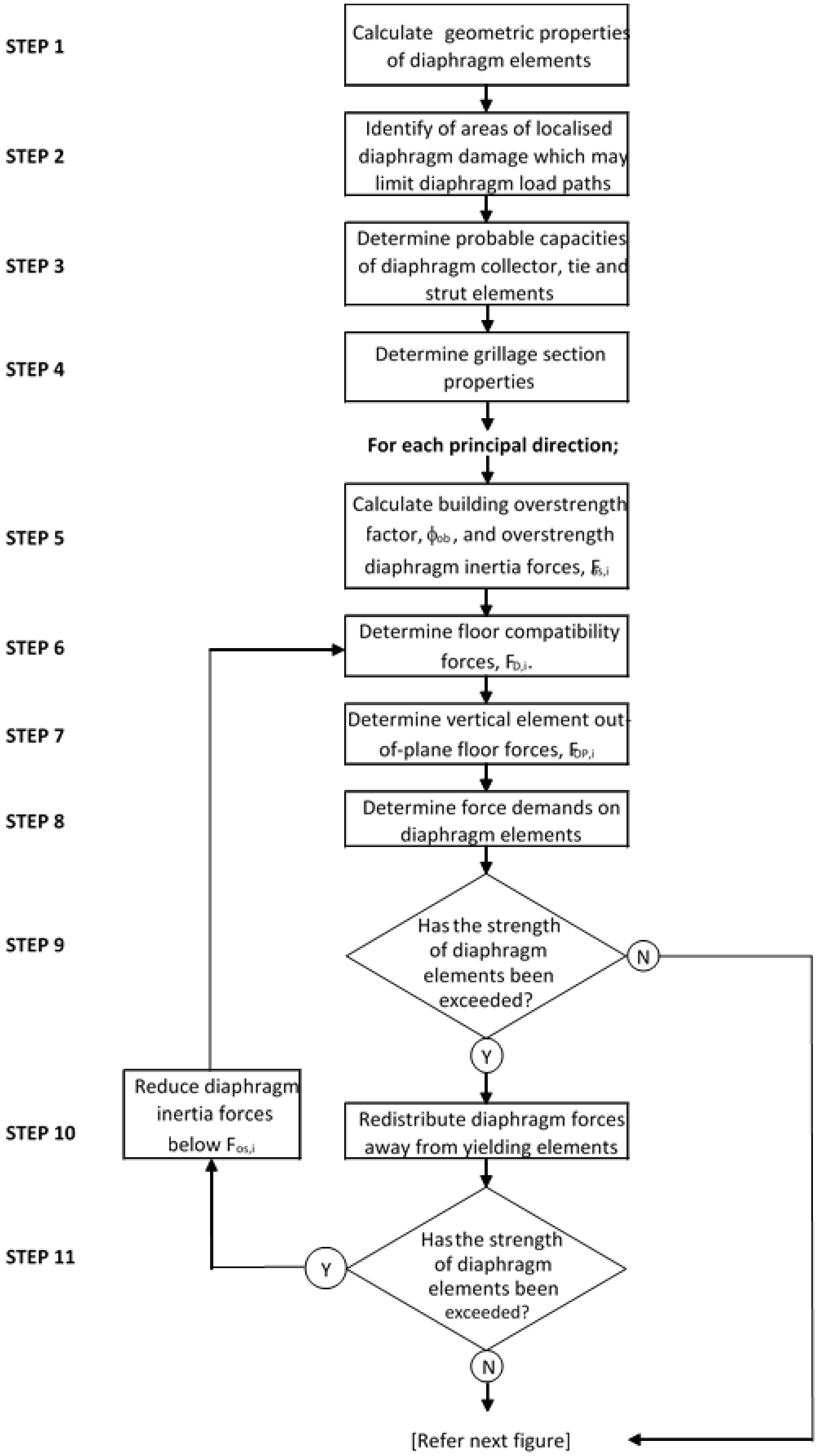


Figure C5.17: Summary of diaphragm assessment procedure – Steps 1 to 11

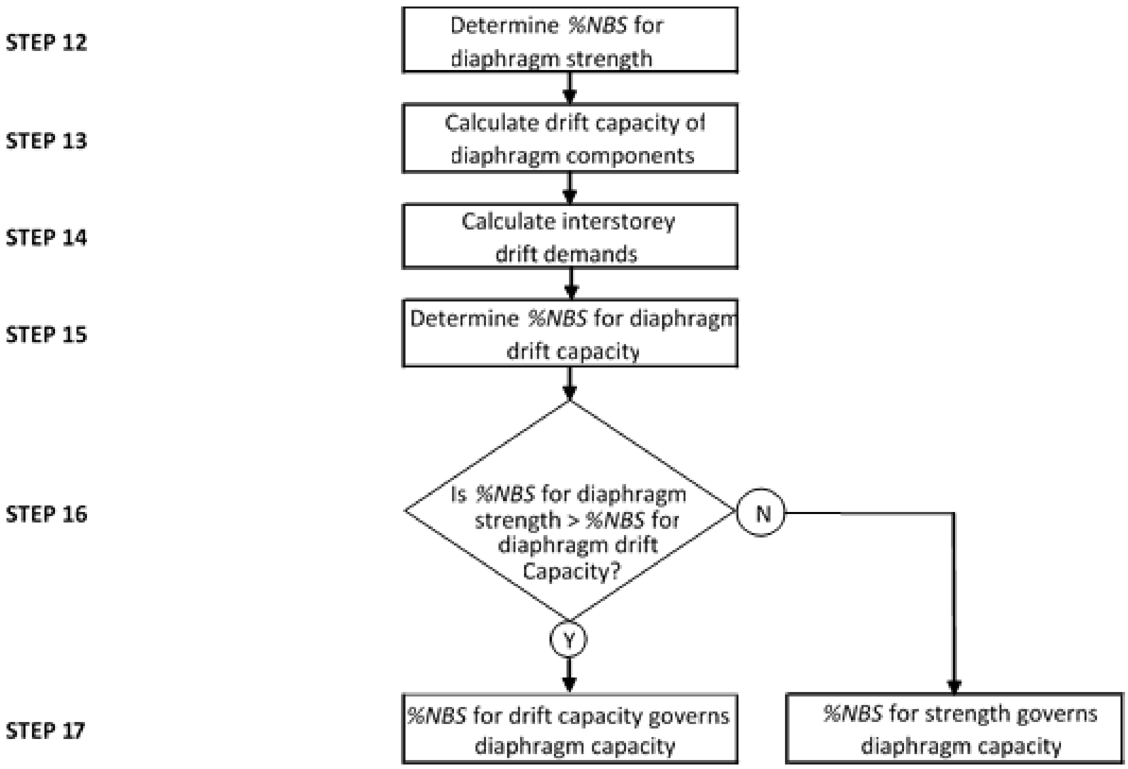


Figure C5.18: Summary of diaphragm assessment procedure – Steps 12 to 17

Step 12

Determine *%NBS* for the diaphragm in terms of strength (refer to Section C5.5.4.4). If the capacity of the diaphragm is greater than the seismic demands calculated using the building overstrength factor, ϕ_{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:

$$\%NBS = 100\frac{0.9R_{\mathrm{prob}}}{K_{\mathrm{d}}R_{\mathrm{E},\mu=1.25}} \qquad \text{...C5.41}$$

where:

R_{prob}	=	probable capacity of diaphragm element calculated in Step 3
$R_{\mathrm{E},\mu=1.25}$	=	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 $S_{\mathrm{p}} = 0.9$
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 the probable inter-storey drift capacity, $\theta_{\text{prob,SC}}$, of diaphragm components. This includes assessing any precast concrete floor units for loss of support and assessing the seismic capacity of the units themselves (refer to Section C5.5.4.3).

Step 14

Calculate inter-storey drift demands, θ_{SD}, in accordance with Section C2 of these guidelines. Section C5.5.4.5 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:

$$\%NBS = 100\,\frac{\theta_{\text{SC}}}{K_{\text{d}}\theta_{\text{SD}}} \qquad \text{…C5.42}$$

where:

$\theta_{\text{prob,SC}}$	=	probable inter-storey drift capacity of diaphragm component
θ_{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.

C5.5.4.3 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 C5.19 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.

Figure C5.19: Observed separation between floor and supporting beam due to beam elongation in 2011 Canterbury earthquakes (Bull)

Appendix C5G 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.

C5.5.4.4 Assessment of diaphragm capacities

The capacity of diaphragm strut and tie elements can be calculated in accordance with Appendix A of NZS 3101:2006 using probable material strengths and a strength reduction factor, ϕ, equal to 1.0. Reduction factors β_n and β_s should be taken as specified in NZS 3101:2006.

C5.5.4.5 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 hollowcore 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 sub-assembly

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 C5.20 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 sub-assembly. 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 often done for simplicity, an erroneous sequence of events might be predicted leading to the potential implementation of an incorrect retrofit strategy.

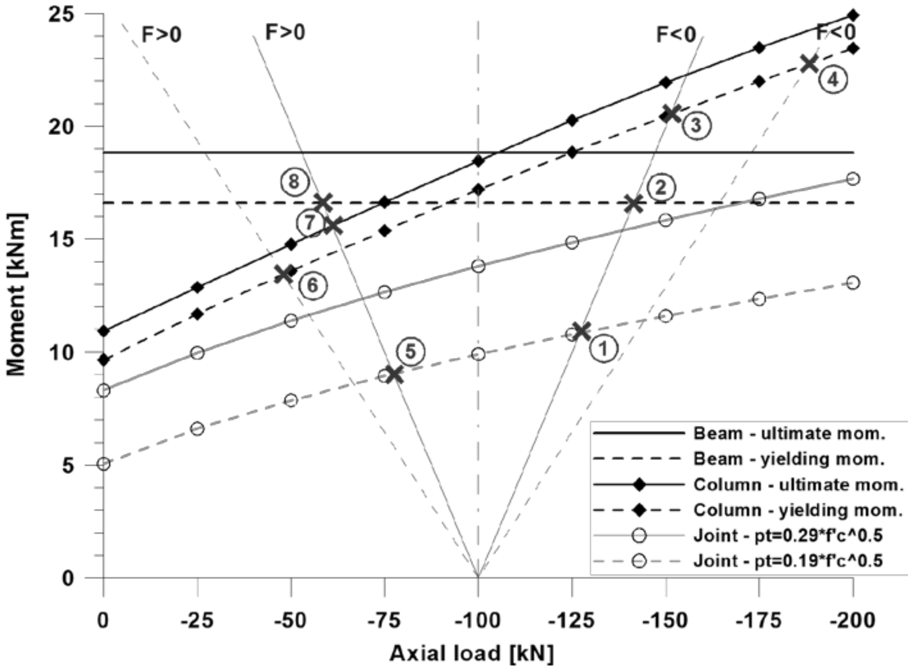
Note:

In the case of the exterior joint shown as an example in Figure C5.20, 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 C5.20) 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 C5.20), 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.



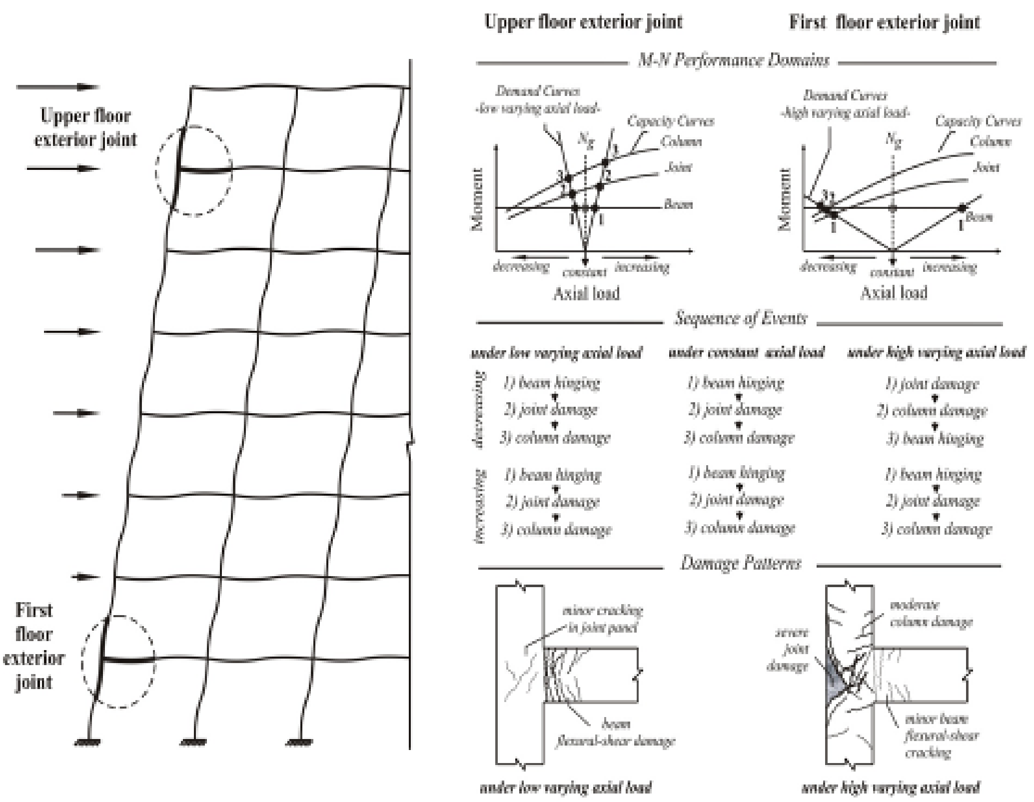
Specimen T1 (as-built)			
Type of lateral force	N°	Event	Lateral force [kN]
Open joint F<0	1	Joint cracking and deterioration starting $p_t = 0.19\sqrt{f_c^{'}}$	-10.94
	2	Beam yielding	-16.59
	3	Upper column yielding	-20.50
	4	Lower column yielding	-22.75
Close joint F>0	5	Joint failure	9.37
	6	Lower column yielding	13.50
	7	Upper column yielding	14.50
	8	Beam yielding	16.59

Figure C5.20: 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)

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 (refer to Figure C5.21). Otherwise, incorrect and non-conservative assessment of the sequence of events can result, which can lead to inadequate – and not necessarily conservative – design of any retrofit intervention.



(a) Laterally loaded frame **(b) Hierarchy of strength and sequence of events for two types of exterior joints**

Figure C5.21: Variation of axial load due to frame sway mechanism and its effects on the hierarchy of strength of beam-column joint subassemblies

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:

$$N = N_g \pm \alpha F \qquad \text{…C5.43}$$

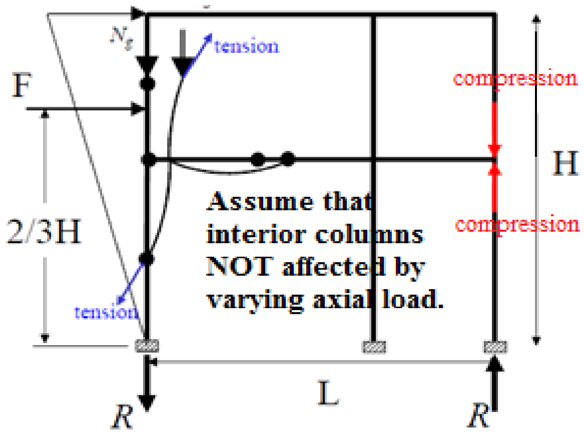
where:

N_g = the axial load due to gravity load
F = the lateral force (base shear capacity), and

α depends on the global geometry of the building (height and total bay length, L, as shown in Figure C5.22).

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 a 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%.



$$F\left(\frac{2}{3}H\right) = RL \Rightarrow R = \frac{2}{3}\frac{H}{L}F \therefore N = N_g \pm \underbrace{\frac{2}{3}\frac{H}{L}}_{\alpha}F$$

Figure C5.22: Example of evaluation of variation of axial load in a frame

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.

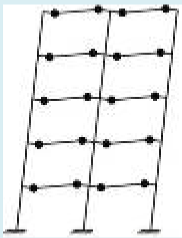
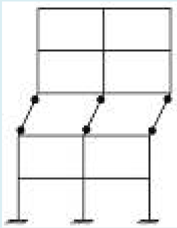
In general, as shown in Table C5.8, upper and lower bounds of the lateral load capacity (i.e. base shear or overturning moment) will be given by a soft-storey mechanism and a beam sway mechanism respectively. Any mixed sidesway mechanisms, including possible shear hinging in the joint, would provide 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 sub-assembly, as follows:

$$OTM = \sum_i M_{\mathrm{col}i} + \left(\sum_x V_{\mathrm{end\ beam},x}\right)L \qquad \ldots C5.44$$

Table C5.8: Upper and lower bounds of frame capacity due to column and beam sway mechanisms, and in-between capacity due to mixed sway mechanism

Upper bound	Lower bound	In between
Beam sidesway mechanism 	Column sidesway mechanism 	Mixed sidesway mechanism 
$OTM,1 = \sum_i M_{coli} + \left(\sum_n V_{\text{end beam,n}}\right)L$ $V_{b,1} = \frac{OTM}{H_{\text{eff,beam sidesway}}}$	$OTM,2 = \sum_i M_{coli}$ $V_{b,2} = \frac{OTM}{H_{\text{eff,col sidesway}}}$	$OTM = \sum_i M_{coli} + \left(\sum_x V^{*}{}_{\text{end beam,x}}\right)L$ $V_{b,3} = \frac{OTM}{H_{\text{eff,mixed sidesway}}}$

Note:

$\sum_i M_{coli}$	=	sum of base column moments
$\sum_n V_{\text{end beam,n}}$	=	sum of end beam shears for all n levels
L	=	frame full span

C5.6.3.2 Beam sidesway mechanism

$$OTM,1 = V_{b,1} * H_{eff} = \sum_i M_{coli} + \left(\sum_n V_{\text{end beam,n}}\right)L \qquad \text{...C5.45}$$

where:

$V_{\text{end beam}}$ = 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

$$OTM,2 = \sum_i M_{coli} = V_b,2 * 0.5h$$

where:

$\sum_i M_{coli}$ = <u>Sum of Moment of the columns at the base</u>

0.5h = point of contraflexure of one floor ...C5.46

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

C5.6.3.4 Mixed mechanism

$$OTM,3 = V_{b,3} * H_{eff} = \sum_i M_{coli} + \left(\sum_x V^*_{\text{end beam},x}\right)L \qquad \text{...C5.47}$$

where:

$V^*_{\text{end beam}}$ 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, $V_{b,2}$, mechanism 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 C5.23.

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.

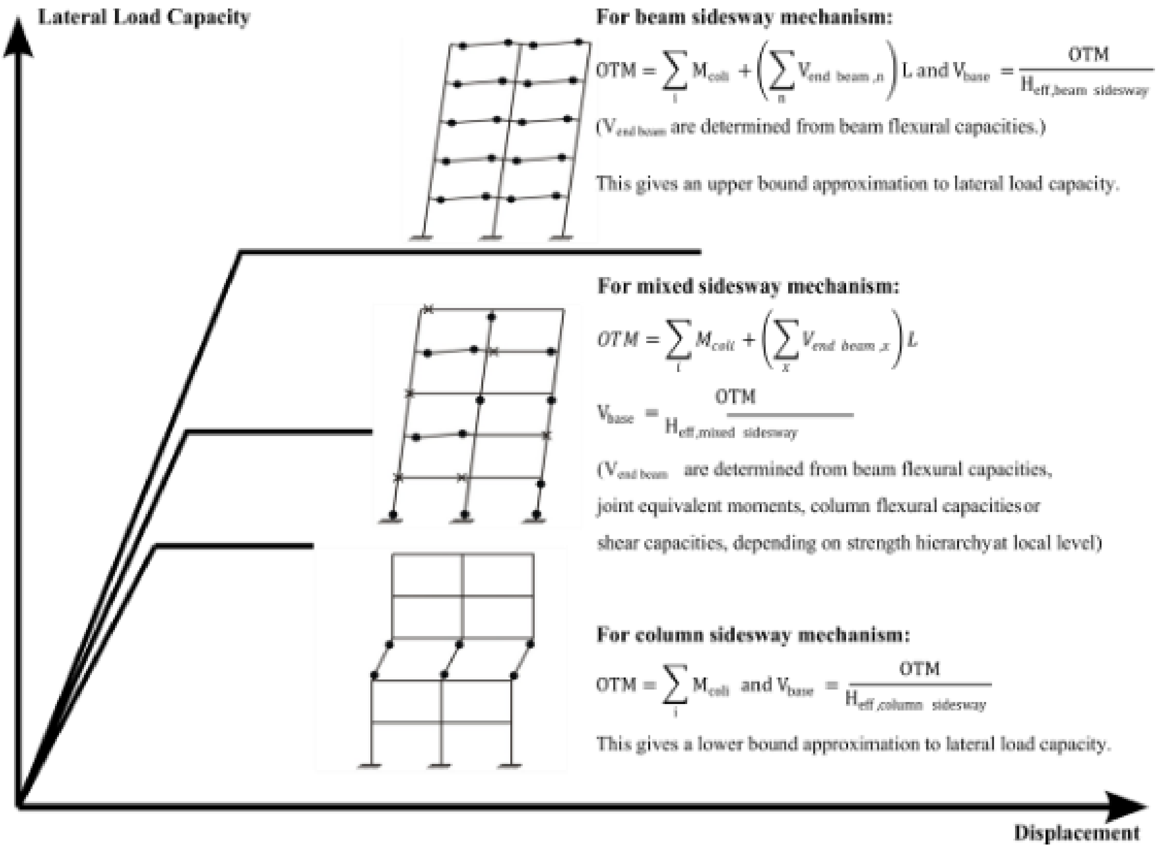


Figure C5.23: Lateral load capacity versus displacement for different global mechanisms

C5.7 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 neglected at a first stage. 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 C2 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 C5.25 with reference to the layout of a wall system shown in Figure C5.24.

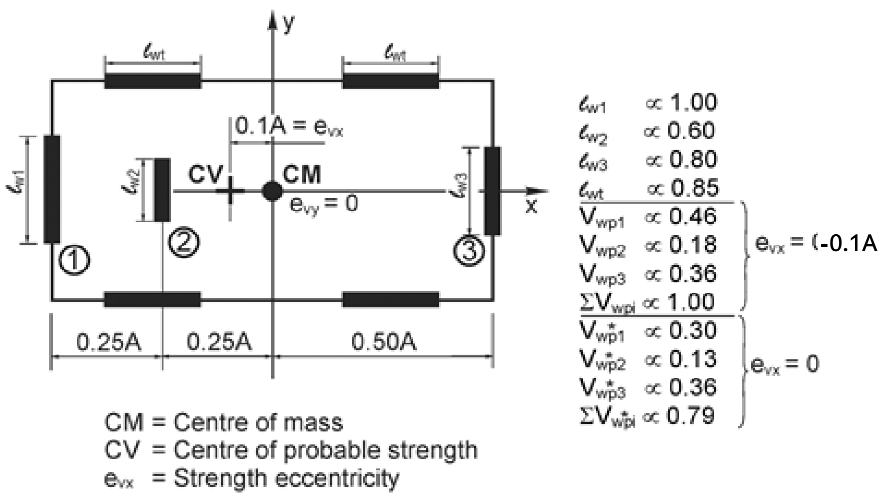
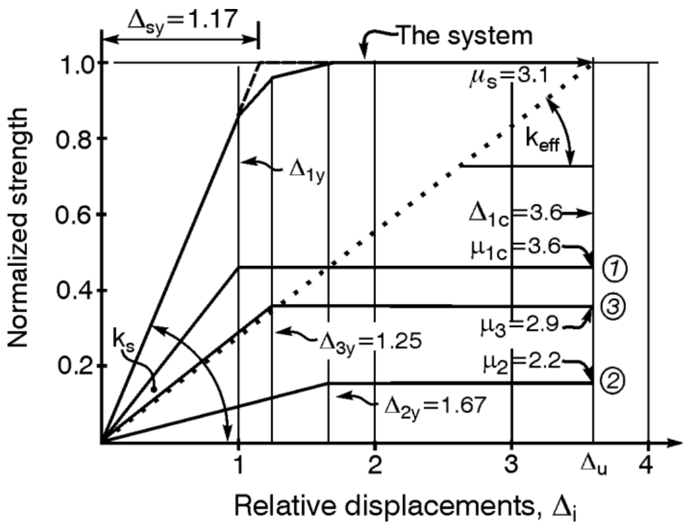


Figure C5.24: (Elastically calculated) torsional effects in a walled building



$U = \Delta$

Figure C5.25: Bilinear idealisation of ductile element and system response for a wall building shown in Figure C5.24

Figure C5.25 shows the global capacity curve and the individual contribution of each wall system.

The relationship between ductilities developed in walls with different 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 is 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 transverse directions should be evaluated.

In any case, the 3D response effects should then be 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.

Regardless 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. Figure C5.26 shows the interaction that may occur between a relatively flexible frame and a wall in a multi-storey building due to the need to achieve displacement compatibility at each level. 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 C5.5.4 for more details).

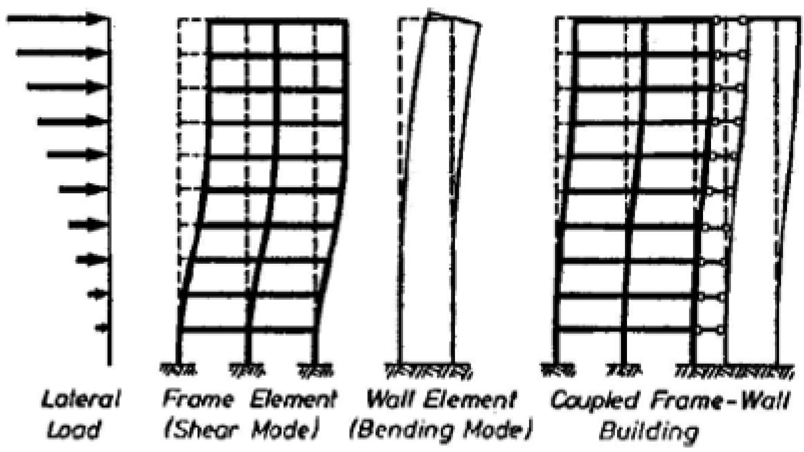


Figure C5.26: Deformation of frame-wall system (Paulay and Priestley, 1992)

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 refer to 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.

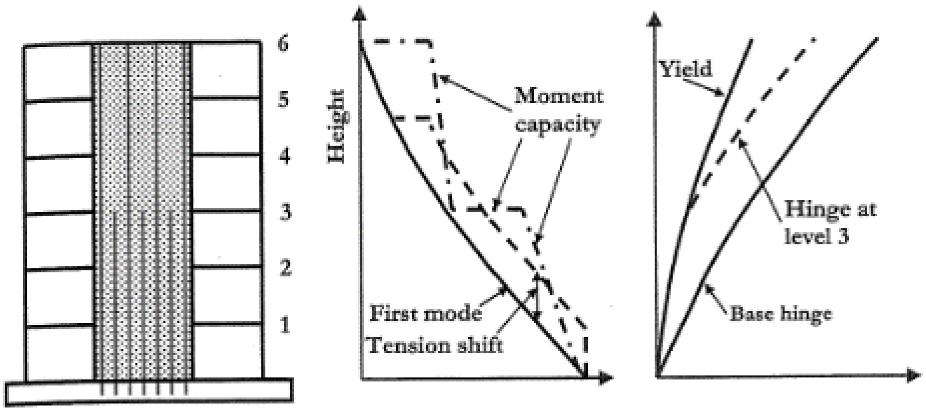
C5.8.2.2 Step by step procedure

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 C5.5.2.1, 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 C5.27, the moment capacity gradually reduces along the height as a consequence of the reduced axial load and longitudinal reinforcement amount.



(a) Structure **(b) Moment profiles** **(c) Displacement profiles**

Figure C5.27: Displacement response of a wall structure (Priestley et al., 2007)

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 C5.27(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 C5.27(b)).

The critical section of the wall can be identified comparing the capacity and demand moment envelope (dash/dot and dashed line in Figure C5.27(b)). If the capacity exceeds the demand at all the levels above the base, such as in the example in Figure C5.27(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 pushover curve of the single-degree-of-freedom (SDOF) system assuming, in first instance, a cantilever wall scheme with $H = H_{\text{eff}}$.

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, $M_{\text{w,b}}$ (subsequently referred to as the wall element).

With this evaluation of the overturning moment capacity of the wall element, $M_{\text{w,b}}$, (refer to Figure C5.29(a)), its probable base shear strength can be estimated from:

$$\sum V_{\text{wp}} = M_{\text{w,b}}/H_{\text{eff}} \qquad \text{…C5.48}$$

The effective height of the wall element, H_{eff}, is given by the approximate position of its point of contraflexure Figure C5.29(a). As a first approximation it can be assumed that $H_{\text{eff}} = 0.67H_w$.

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 $H_{\text{eff}} < 0.67H_w$.

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 ULS conditions can be computed according to Section C5.5.2.2.

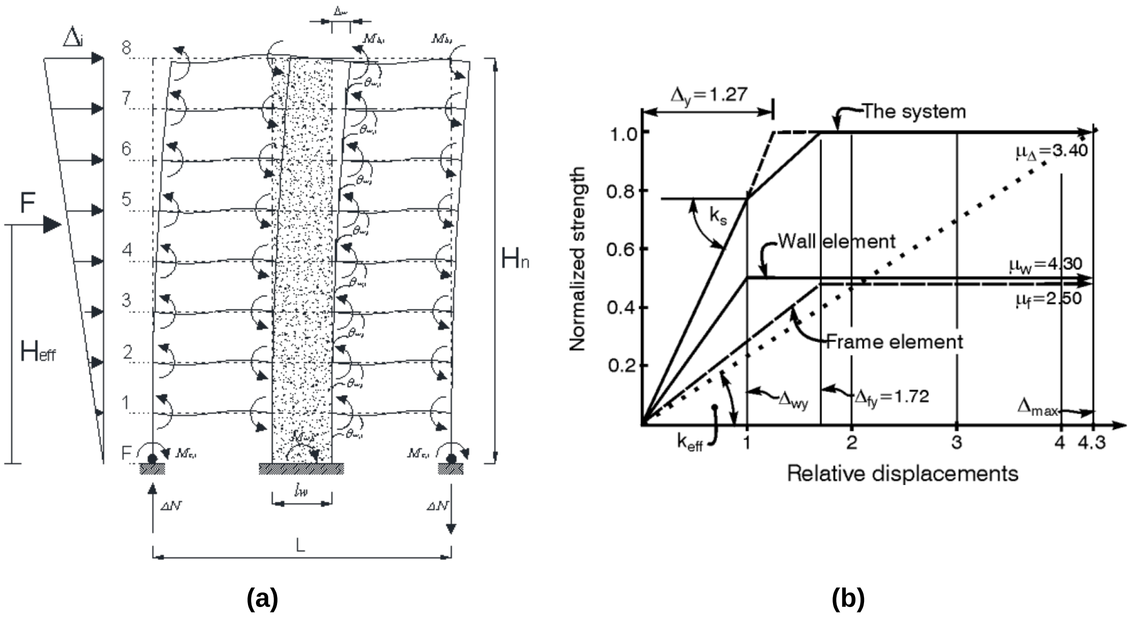
Step 2 Establish the post-elastic mechanism of frames and their contribution to lateral force resistance

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 C5.28.

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.



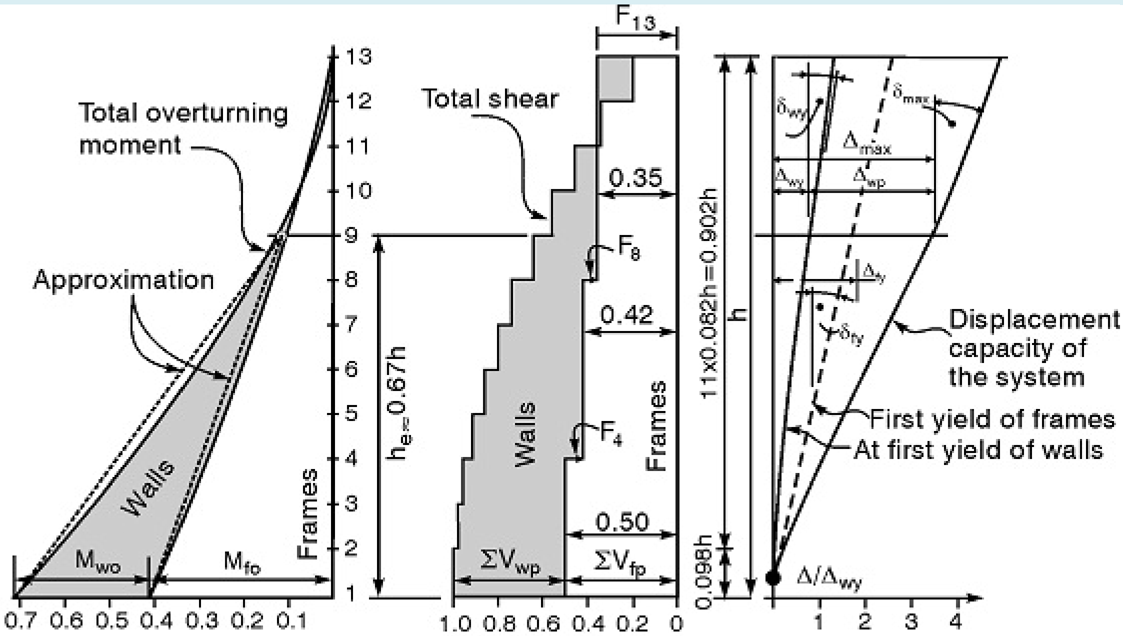
(a) **(b)**

Figure C5.28: Contribution of frame and wall to the global force-displacement capacity curve

Note:

Figures C5.28 and C5.29 illustrate the procedure described at Step 2, with a kinematically admissible sway mechanism. Plastic hinges introduce a total moment of $\sum M_{\mathrm{pi}}$ 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.



(a) Normalised overturning moments ($M/hV_{\mathbf{b}}$) **(b) Normalised storey shears ($V_{s/}V_{\mathbf{b}}$)** **(c) Displacement profiles**

Figure C5.29: 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 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 C5.28(b)).

Note:

Figure C5.28(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 Section C2 on mixed ductility systems. Note that this figure represents the expected behaviour of the schematic dual system shown in Figure C5.27 (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 C5.29(b) shows, an approximately equal contribution (50-50) to the probable base shear strength of the dual system, $V_{\mathrm{dual,p}}$, 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:

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

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

$$k_{\mathrm{w}} = V_{\mathrm{wp}}/\Delta_{\mathrm{wy}} = 0.5/1.0 = 0.5$$

$$k_{\mathrm{f}} = V_{\mathrm{fp}}/\Delta_{\mathrm{fy}}\ 0.5/1.72 = 0.29$$

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

$$\Delta_{\mathrm{y}} = V_{\mathrm{dual,p}}/(K_{\mathrm{w}} + K_{\mathrm{f}}) = 1.00/(0.5 + 0.29) = 1.27 \text{ displacement units}$$

The bilinear idealisation of the force-displacement curve for frame, wall and dual system behaviour, shown in Figure C5.29(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 (2006); 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 and Bibliography

ACI 318-63 (1963). *Building code requirements for reinforced concrete* (ACI 318-63), American Concrete Institute (ACI), Detroit, Michigan.

ACI 318-71 (1972). *Building code requirements for reinforced concrete* (ACI 318-71), American Concrete Institute (ACI), Detroit, Michigan.

Akguzel, U. (2011). *Seismic performance of FRP retrofitted exterior RC beam-column joints under varying axial load and bidirectional loading*, PhD Thesis, Department of Civil and Natural Resource Engineering, University of Canterbury, 2011.

Akguzel, U. and Pampanin S. (2010). *Effects of variation of axial load and bidirectional loading on seismic performance of GFRP retrofitted reinforced concrete exterior beam-column joints,* ASCE Journal of Composites for Construction, 14(1):94-104, Jan-Feb 2010.

Andriono, T. and Park, R. (1986). *Seismic design considerations of the properties of New Zealand manufactured steel reinforcing bars,* Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 19, No. 3 213-246, September 1986.

AS/NZS 4671:2001. *Steel reinforcing materials*. Standards Australia/Standards New Zealand.

ASCE 41-13 (2014). *Seismic evaluation and retrofit of existing buildings*, American Society of Civil Engineers, and Structural Engineering Institute, Reston, Virginia, USA.

ATC 40 (1996). *Seismic evaluation and retrofit of concrete buildings*, Applied Technology Council, Redwood City, California, USA, Vol 1 & 2, report SSC 96-01, Nov 1996.

Bech, D., Cordova, P., Tremayne, B., Tam, K., Weaver, B., Wetzel, N., Parker, W., Oliver, L. and Fisher, J. (2014). *Common structural deficiencies identified in Canterbury buildings and observed versus predicted performance*, Earthquake Spectra, Journal of Earthquake Engineering Research Institute, Vol. 30, No. 1, 277-306. 2014.

Berry, M.P. and Eberhard, M.O. (2005). *Practical performance model for bar buckling*, Journal of Structural Engineering, ASCE.

Boys, A., Bull, D. K. and Pampanin, S. (2008). *Seismic performance assessment of inadequately detailed reinforced concrete columns*, New Zealand Society of Earthquake Engineering (NZSEE) Conference. Wairakei, NZ.

Blume, J.A., Newmark, N.M. and Corning, L. (1961). *Design of multi-storey reinforced concrete buildings for earthquake motions,* Portland Cement Association, Chicago, IL.

BS165-1929 (1929). *Hard drawn steel wire for concrete reinforcement*, British Standards Institution, London, UK.

BS785-1938 (1938). *Rolled steel bars and hard drawn steel wire for concrete reinforcement*, British Standards Institution, London, UK.

Carr, A.J. (2005). *RUAUMOKO Users Guide*, Department of Civil Engineering, University of Canterbury.

CEB-1964 (1964). *Recommendations for an international code of practice for reinforced concrete*, American Concrete Inst. (ACI) & Cement and Concrete Assoc. (CCA), London, UK.

CEN (2004). European Standard EN 1992-1-1: Eurocode 2: *Design of concrete structures - Part 1-1: General rules and rules for buildings*, Comite Europeen de Normalisation, Brussels.

CERC (2012) Canterbury Earthquake Royal Commission websites, Canterbury Earthquake Royal Commission (CERC) website. Available at: http://canterbury.royalcommission.govt.nz.

Chai, Y.H. and Elayer, D.T. (1999). *Lateral stability of reinforced concrete columns under axial reversed cyclic tension and compression*, ACI Structural Journal 96:5, 780-789.

Chapman, H.E. (1991). *Seismic retrofitting of highway bridges*, Bulletin of New Zealand Society for Earthquake Engineering, Vol 24, No. 2, 186–201.

Cheung, P.C., Paulay, T. and Park, R. (1991). *Mechanisms of slab contributions in beam - column sub assemblages. Design of beam-column joints for seismic resistance*. Special Publication SP-123 American Concrete Institute; 259-289.

CP 114:1957 (1957). *The structural use of concrete in buildings*, British Standards Institution (BSi), London, UK.

Cuevas, A. et al., (2015). *Details of beams,* Residual capacity report.

Cull, J.E.L. (Chairman) (1931). *Report of the Building Regulations Committee,* Report to New Zealand House of Representatives, H-21.

Dashti, F., Dhakal, R., Pampanin, S., (2015a). *Comparative in-plane pushover response of a typical RC rectangular wall designed by different standards*, Earthquakes and Structures 7(5); 667-689. . http://dx.doi.org/10.12989/eas.2014.7.5.667.

Dashti, F., Dhakal, R.P. and Pampanin, S. (2015b). *Development of out-of-plane instability in rectangular RC structural walls,* Rotorua, New Zealand: 2015 New Zealand Society for Earthquake Engineering Annual Conference (NZSEE).

Dashti, F., Dhakal, R., Pampanin, S., (2016), *Simulation of out-of-plane instability in rectangular RC structural walls subject to in-plane loading*, ASCE Journal of Structural Engineering, under publication.

Di Franco, M.A., Mitchell, D. and Paultre, P. (1995). *Role of spandrel beams on response of slab beam-column connections,* ASCE Journal of Structural Engineering, 121(3):408-419.

Durrani, A.J. and Zerbe, H.E. (1987). *Seismic resistance of R/C exterior connections with floor slab*, ACI Structural Journal. 113(8):1850-1864.

Ehsani, M.R. and Wight, J. (1985). *Effect of transverse beams and slab on behaviour of reinforced concrete beam-to-column connections*, ACI Structural Journal. Mar-Apr 195; 82(2):188-195.

Elwood, K.J. and Moehle, J.P. (2005). *Drift capacity of reinforced concrete columns with light transverse reinforcement*, Earthquake Spectra, February 2005, Vol. 21, No. 1, pages 71-89.

Elwood, K.J. and Moehle, J.P. (2005). *Axial capacity model for shear-damaged columns*, ACI Structural Journal. 102:578-587.

Fabbrocino, G., Verderama, G.M., Manfredi, G. and Cosenza, E. (2002). *Experimental behaviour of smooth bars anchorages in existing RC buildings*, Proc. of 1st fib Congress, Osaka, Japan, 259-268.

Fabbrocino, G., Verderame, G.M. and Manfredi, G. (2002). *Experimental behaviour of straight and hooked smooth bars anchorage in existing R.C. buildings*, Proc. of 12th European Conference on Earthquake Engineering, Elsevier, London, UK.

FEMA 440(2005). *Improvement of nonlinear static seismic analysis procedures*, Federal Emergency Management Agency.

Fenwick. R. and MacRae, G.A. (2009). *Comparison of New Zealand standards used for seismic design of concrete buildings*, Bulletin of New Zealand Society of Earthquake Eng. Sept 2009; 42(3):187-203.

Fenwick, R.C., Bull, D.K. and Gardiner, D. (2010). *Assessment of hollow-core floors seismic performance*, Research Report 2010-02, Department of Civil and Natural Resources Engineering, University of Canterbury. http://hdl.handle.net/10092/4211.

Fleischman, R, Restrepo, J. I., Pampanin, S. Maffei, J.R., Seeber, K., Zahn, F.A., (2014) *Damage evaluations of precast concrete structures in the 2010-2011 New Zealand earthquakes*, Earthquake Spectra, Journal of Earthquake Engineering Research Institute, Special Issue, doi: http://dx.doi.org/10.1193/031213EQS068M, Vol.30, No. 1, 277-306.

Ford, C.R. (1926). *Earthquakes and building constructions*, Whitcombe and Tombs Ltd, Auckland, NZ.

Gardiner, D.R., Bull, D.K. & Carr, A. (2008). *Investigation of the magnitude of inertial and transfer forces in floor diaphragms during seismic shaking*, in Proceedings, the 14th World Conference on Earthquake Engineering, Beijing, China, 2008.

Gardiner, D.R. (2011). *Design recommendations and methods for reinforced concrete floor diaphragms subjected to seismic forces*, PhD Thesis, Department of Civil and Natural Resource Engineering, University of Canterbury, 2011.

Genesio, G. (2012). *Seismic assessment of RC exterior beam-column joints and retrofit with haunches using post-installed anchors*, MD thesis, Institut für Werkstoffe im Bauwesen der Universität Stuttgart, 2012.

Hakuto, S., Park, R. and Tanaka, H. (1995). *Retrofitting of reinforced concrete moment resisting frames*, Research Report 95–4, Department of Civil Engineering, University of Canterbury, NZ.

Hakuto, S., Park, R. and Tanaka, H. (1999). *Effect of deterioration of bond of beam bars passing through interior beam-column joints on flexural strength and ductility*, Structural Journal of American Concrete Institute 96(5): 858–64, September–October.

Hakuto, S., Park, R. and Tanaka, H. (2000). *Seismic load tests on interior and exterior beam-column joints with substandard reinforcing details*, Structural Journal of American Concrete Institute 97(1): 11–25, January–February.

Henry, R.S. (2013). *Assessment of minimum vertical reinforcement limits for RC walls*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 46, No. 2, June 2013.

Hollings, J.P. (1969). *Reinforced concrete seismic design*, Bulletin of New Zealand Society for Earthquake Engineering, Vol. 2, No. 3 217-250.

Holmes. (2015). Practice Note 8.1 *Modelling diaphragm force distributions – simple grillage method*, Holmes Consulting Group, Christchurch, New Zealand, Unpublished.

Hrennikoff, A. (1941). *Solution of problems of elasticity by the framework method*, Journal of Applied Mechanics, December 1941, 7 pp.

Kam, W.Y. (2011), *Selective weakening and post-tensioning for the seismic retrofit of non-ductile R.C. frames*, PhD Thesis, Department of Civil and Natural Resources Engineering, University of Canterbury, 2011.

Kam, W.Y. and Pampanin, S. (2011). *General building performance in the Christchurch CBD: a contextual report* (prepared for the Department of Building and Housing (DBH)) as part of the Technical Investigations into the Performance of Buildings in the Christchurch CBD in the 22 February Christchurch Aftershock. University of Canterbury: Christchurch.

Kam, W.Y. and Pampanin, S. (2012). *Revisiting performance-based seismic design in the aftermath of the Christchurch 2010-2011 earthquakes: raising the bar to meet societal expectations*, Lisbon, Portugal: 15th World Congress on Earthquake Engineering (15WCEE).

Kam, W.Y., Pampanin, S. and Elwood, K. (2011). *Seismic performance of reinforced concrete buildings in the 22 February Christchurch (Lyttelton) earthquake,* Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 44, No. 4, 239-278, December 2011.

Kam, W.Y., Pampanin, S., Dhakal, R.P., Gavin, H. and Roeder, C.W. (2010). *Seismic performance of reinforced concrete buildings in the September 2010 Darfield (Canterbury) earthquakes*, Bulletin of New Zealand Society for Earthquake Engineering, Vol 43, No. 4, 340-350, December 2010.

Kowalsky, M.J. and Priestley, M.J.N. (2000). *Improved analytical model for shear strength of circular reinforced concrete in seismic regions*, ACI Structural Journal. May-June 2000; 97(3):388-396.

Krolicki, J., Maffei, J. and Calvi, G.M. (2011). *Shear strength of reinforced concrete walls subjected to cyclic loading*, Journal of Earthquake Engineering, 15:S1, 30-71.

Liu, A. and Park, R. (1998). *Seismic load tests on two interior beam-column joints reinforced by plain round bars designed to pre-1970s seismic codes,* Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 31, No. 3, 164–76.

Liu, A. and Park, R. (2001). *Seismic behaviour and retrofit of pre-1970s as-built exterior beam-column joints reinforced by plain round bars*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 34, No. 1, 68–81.

MacGregor, G.J. (1997). *Reinforced concrete mechanics and design*, 3rd Edition. Prentice-Hall, Englewood Cliffs, NJ.

Mander, J.B., Priestley, M.J.N. and Park, R. (1988). *Observed stress-strain behaviour of confined concrete,* Journal of Structural Engineering, American Society of Civil Engineers 114(8): 1827–49.

Mander, J.B., Priestley, M.J.N. and Park, R. (1988). *Theoretical stress-strain model for confined concrete*. Journal of Structural Engineering, American Society of Civil Engineers 114(8): 1804–26.

Magenes, G. and Pampanin, S. (2004). *Seismic response of gravity-load design frames with masonry infills*, Proceedings of 13th World Conference on Earthquake Engineering, Vancouver, Canada. Paper No. 4004.

Marriott, D. (2009). *The development of high performance post-tensioned rocking systems for the seismic design of structures*, PhD Thesis, Department of Civil and Natural Resource Engineering, University of Canterbury, 2009.

Megget, L.M. (2006). *From brittle to ductile: 75 years of seismic design in New Zealand*, Proceedings of NZSEE 2006 Conference, NZSEE, Napier, NZ, p. 15.Keynote Address.

MOW-NZ (1968). *Manual of draughting practice: civil engineering*. Ministry of Works, Wellington, NZ.

MOW-NZ (1970). PW81/10/1: *Code of practice, Design of public buildings*, Office of Chief Structural Engineer, Ministry of Works, Wellington, NZ.

Niroomandi, A., Pampanin, S. and Dhakal, R. (2015). *The history of design guidelines and details of reinforced concrete column in New Zealand*, Proceedings of the NZSEE15, Rotorua, NZ.

NZS 197:1949. *Rolled steel bars and hard drawn steel wire,* NZS 197:1949. Standards Association of New Zealand, Wellington, NZ.

NZS 1170.5:2004. *Structural design actions, Part 5: Earthquake actions – New Zealand,* NZS 1170.5:2004. Standards New Zealand, Wellington, NZ.

NZS 1693:1962. *Deformed steel bars of structural grade for reinforced concrete*. NZS 1693:1962. Standards Association of New Zealand, Wellington, NZ.

NZS 1879:1964. *Hot rolled steel bars of HY60 grade (60,000 psi) for reinforced concrete*, NZS 1879:1964. Standards Association of New Zealand, Wellington, NZ.

NZS 1900.8:1965. *Model building bylaw: Basic design loads and commentary*, NZS 1900. Standards Association of New Zealand, Wellington, NZ.

NZS 3101:1970P. *Code of practice for the design of concrete structures*, NZS 3101:1970 (Provisional). Standards Association of New Zealand, Wellington, NZ.

NZS 3101:1982. *Code of practice for the design of concrete structures*. NZS 3101:1982. Standards Association of New Zealand, Wellington, NZ.

NZS 3101:1995. *Concrete structures standard*, NZS 3101:1995, Volume 1 Code of Practice and Volume 2 Commentary. Standards New Zealand, Wellington, NZ.

NZS 3101:2006. *Concrete structures standard*, NZS 3101:2006. Standards New Zealand, Wellington, NZ.

NZS 3402P:1973. *Hot rolled steel bars for the reinforcement of concrete*, NZS 3402P:1973. Standards Association of New Zealand, Wellington, NZ.

NZS 3402:1989. *Steel bars for the reinforcement of concrete*, NZS 3402:1989. Standards New Zealand, Wellington, NZ.

NZS 3423P:1972. *Hot rolled plain round steel bars of structural grade for reinforced concrete,* NZS 3423P:1972. Standards Association of New Zealand, Wellington, NZ.

NZS 4203:1976. *Code of practice for general structural design and design loading for buildings*, NZS 4203:1976. Standards Association of New Zealand, Wellington, NZ.

NZS 4203:1984. *Code of practice for general structural design and design loadings for buildings*, NZS 4203:1984. Standards Association of New Zealand, Wellington, NZ.

NZS 4203:1992. *General structural design and design loadings for buildings*, NZS 4203:1992. Standards New Zealand, Wellington, NZ.

NZSS 95:1935. *New Zealand standard model buildings by-laws*, NZSS 95:1935. New Zealand Standards Institute, Wellington, NZ.

NZSS 95:1939. *New Zealand standard code of buildings by-laws*, NZSS 95:1939. New Zealand Standards Institute, Wellington, NZ.

NZSS 95:1955. *New Zealand standard - Model building by-laws*, Part IV and V. NZSS 95:1955. New Zealand Standard Institute, Wellington, NZ.

Pampanin, S. (2006). *Controversial aspects in seismic assessment and retrofit of structures in modern times: Understanding and implementing lessons from ancient heritage,* Bulletin of New Zealand Society for Earthquake Engineering, Vol. 39, No. 2, 120-133.

Pampanin, S. (2009). *Alternative performance-based retrofit strategies and solutions for existing RC buildings,* Geotechnical, Geological and Earthquake Engineering. Vol. 10. In: Ilki, A., Karadogan, F., Pala, S. and Yuksel, E., editors. *Seismic risk assessment and retrofitting with special emphasis on existing low rise structures*. Springer, Netherlands; 267-295.

Pampanin, S., (2010). *Retrofit solutions for pre-1970 R.C. buildings: an overview of latest research development in New Zealand*, European Conference in Earthquake Engineering, Ohrid, Macedonia,

Pampanin, S., Calvi, G.M. and Moratti, M. (2002). *Seismic behaviour of RC beam-column joints designed for gravity loads,* Proceedings of 12th European Conference on Earthquake Engineering, London, UK. Paper 726.

Pampanin, S., Kam, W.Y., Akguzel, U., Tasligedik, A.S. and Quintana-Gallo, P. (2012). *Seismic performance of reinforced concrete buildings in the Christchurch central business district (Natural Hazard Platform Recovery Project)*, University of Canterbury, Christchurch, NZ.

Pampanin, S., Magenes, G. and Carr, A. (2003). *Modelling of shear hinge mechanism in poorly detailed RC beam-column joints*, Proceedings of the Concrete Structures in Seismic Regions: fib 2003 Symposium, Federation International du Beton, Athens, Greece, Paper No. 171.

Park, R. (1992). *Seismic assessment and retrofit of concrete structures: United States and New Zealand developments*, Proceedings of Technical Conference of New Zealand Concrete Society, Wairakei, New Zealand, 18–25.

Park, R. (1996). *A static force-based procedure for the seismic assessment of existing reinforced concrete moment resisting frames*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 30, No. 3, 213-226.

Park, R. and Paulay, T. (1975). *Reinforced concrete structures*. John Wiley and Sons, New York.

Park, R., Billings, I.J., Clifton, G.C., Cousins, J., Filiatrault, A., Jennings, D.N., Jones, L.C.P., Perin, N.D., Rooney, S.L., Sinclair, J., Spurr, D.D., Tanaka, H. and Walker, G. (1995). *The Hyogo-ken Nanbu earthquake (the Great Hanshin earthquake) of 17 January 1995*, Report of the NZNSEE Reconnaissance Team. Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 28, No. 1, 1-98.

Paulay, T., Zanza, T.M. and Scarpas, A. (1981). *Lapped splice in bridge piers and in columns or earthquake resisting reinforced concrete frames*, UC Research Report 81-6. Department of Civil Engineering, University of Canterbury, Christchurch, NZ.

Paulay, T. and Priestley, M.J.N. (1992). *Seismic design of reinforced concrete and masonry buildings*, John Wiley and Sons, New York.

Paulay, T. (1993). *Simplicity and confidence in seismic design*, John Wiley and Sons, New York.

Paulay, T. and Restrepo, J. I. (1998). *Displacement and ductility compatibility in buildings with mixed structural systems*, Journal of the Structural Engineering Society, New Zealand. 11(1): 7-12.

Paulay, T. (2000). *Principles of displacement compatibility*, Journal of the Structural Engineering Society, New Zealand. Vol. 13(2):14-21.

Paulay, T. (2001a). *A re-definition of the stiffness of reinforced concrete elements and its implications in seismic design*, Structural Engineering International 11(1): 36-41.

Paulay, T. (2001b). *Seismic response of structural walls: recent developments*, Canadian Journal of Civil Engineering. 28: 922-937.

Paulay, T. (2002). *An estimation of displacement limits for ductile systems*, Earthquake Engineering and Structural Dynamics 31:583-599.

Pekcan, G., Mander, J.B. and Chen, S.S. (1999). *Fundamental considerations for the design of nonlinear viscous dampers*, Earthquake Engineering Structural Dynamics 28, 1405-1425, 1999.

Presland, R.A. (1999). *Seismic performance of retrofitted reinforced concrete bridge piers*, PhD Thesis, Department of Civil Engineering, University of Canterbury.

Priestley, M.J.N. and Park, R. (1987). *Strength and ductility of concrete bridge columns under seismic loading*, Structural Journal of American Concrete Institute 84(1): 61–76.

Priestley, M.J.N. (1988). *Brief comments on elastic flexibility of reinforced concrete frames and significance to seismic design,* Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 31. No. 4 246-259.

Priestley, M.J.N., Verma, R. and Xiao, Y. (1994). *Seismic shear strength of reinforced concrete columns*, Journal of Structural Engineering, American Society of Civil Engineers 120(8): 2310–29.

Priestley, M.J.N. (1995). *Displacement-based seismic assessment of existing reinforced concrete buildings*, Proceedings of Pacific Conference on Earthquake Engineering, Melbourne, Australia, 225-244.

Priestley, M.J.N., Seible, F. and Calvi, G.M. (1996). *Seismic design and retrofit of bridges*, John Wiley and Sons, New York.

Priestley, M.J.N. and Kowalsky, M.J. (2000). *Direct displacement-based seismic design of concrete buildings,* Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 33, No. 4 421–443, December 2000..

Priestley, M.J.N., Calvi, G.M. and Kowalsky, M.J. (2007). *Displacement-based seismic design of structures*, IUSS Press, Pavia, Italy.

Quintana-Gallo, P. (2014). *The nonlinear dynamics involved in the seismic assessment and retrofit of reinforcement concrete buildings*, PhD Thesis, Department of Civil and Natural Resource Engineering, University of Canterbury, 2014.

Rodriguez, M. and Park, R. (1991). *Repair and strengthening of reinforced concrete buildings for earthquake resistance*, Earthquake Spectra 7(3): 439–59.

Scott, B.D., Park, R. and Priestley, M.J.N. (1982). *Stress-strain behaviour of concrete confined by overlapping hoops at low and high strain rates*, Journal of the American Concrete Institute Proceedings 79(1): 13–27.

SEAOC (1966). *Recommended lateral force requirements*, Structural Engineers Association of California (SEAOC), Sacramento, CA.

SEAOC (1973). *Recommended lateral force requirements and commentary*, Structural Engineers Association of California (SEAOC), Sacramento, CA.

SESOC (2011). *Practice note: Design of conventional structural systems following the Canterbury earthquakes,* submission by Structural Engineering Society of New Zealand, NZ.

Sezen, H., Hookham, C., Elwood, K., Moore, M., and Bartlett, M. (2011). *Core testing requirements for seismic evaluation of existing reinforced concrete structures*, Concrete International, American Concrete Institute, November 2011.

Shibata, A. and Sozen, M.A. (1976). *Substitute structure method for seismic design in reinforced concrete*, Journal of Structural Engineering, American Society of Civil Engineers. 102(1), 1-18.

Shin, M. and LaFave, J. (2004). *Reinforced concrete edge beam– column–slab connections subjected to earthquake loading,* Magazine of Concrete Research 56(5): 273–291.

Sritharan, S., Beyer, K., Henry, R.S., Chai, Y.H. Kowalsky, M. and Bull, D. (2014). *Understanding poor seismic performance of concrete walls and design implications,* Earthquake Spectra, 30(1), 307-334.

Stirrat, A.T., Gebreyohaness, A.S., Jury, R.D. and Kam, W.Y. (2014). *Seismic performance assessment of non-ductile columns,* Proceedings of the 2014 New Zealand Society for Earthquake Engineering Conference, Auckland, New Zealand, 21-23 March 2014, Paper O1. 17 pages..

Sullivan, T.J., Priestley, M.J.N. and Calvi, G.M. Editors (2012). *A model code for the displacement-based seismic design of structures*, DBD12, IUSS Press, Pavia, ISBN 978-88-6198-072-3, 105 pages.

Tasligedik, A.S., Kam, W.Y., Akguzel, U. and Pampanin, S. (2016). *Calculation of strength hierarchy at reinforced concrete beam-column joints: from experimental studies into structural engineering applications*, Journal of Earthquake Engineering, under review.

Tasligedik, A.S., Pampanin, S. and Palermo, A. (2015). *Low-damage seismic solutions for non-structural drywall partitions*, Bulletin of Earthquake Engineering 13(4): 1029-1050.

Voon, K.C., Ingham, J.M., *Experimental in-plane shear strength investigation of reinforced concrete masonry walls*, Journal of Structural Engineering, ASCE, 132(3), pages 400-408.

Wallace, J.L. (1996). *Behaviour of beam lap splices under seismic loading,* Master of Engineering Thesis, Department of Civil Engineering, University of Canterbury.

Wallace, J.L., Segura, C.L., Arteta, C.A. and Moehle, J.P. (2016). *Deformation capacity of thin reinforced concrete shear walls,* Proceedings of Technical Conference of New Zealand Concrete Society, Christchurch, NZ.

Yoshimura, M. (2008). *Formulation of post-peak behaviour of old reinforced concrete columns until collapse,* 14th World Conference on Earthquake Engineering. 12-17 October 2008. Beijing, China.

Appendix C5A History of New Zealand Concrete Design Standards and Code-based Reinforcing Requirements

C5A.1 Introduction

This appendix provides a historical overview of New Zealand's concrete design standards. It also summarises the history of the country's code-based reinforcement requirements for:

- beams
- columns
- beam-column joints, and
- walls.

C5A.2 Evolution of Concrete Design Standards

The following table sets out key milestones in the development of New Zealand concrete design standards, from pre-1957 to the present day.

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

Period	Loading Standard	Concrete Standard	Major changes
Pre-1957	1935 Model Bylaws	No seismic provisions	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.
1957-1964	NZSS 95 - Pt IV Basic Loads to be used and methods of application (1955)	UK concrete Code of Practice, CP114:1957 (No seismic provisions) and NZSS 95, Pt V (1939)	Section properties of members were permitted to be based on gross sections, transformed un-cracked sections, or transformed cracked sections (Fenwick and MacRae, 2009).
1964-1968/71	NZSS 1900 Basic Design Loads Chapter 8 (1965)	Design and Construction, Concrete, Chapter 9.3, 1964 (No seismic provisions)	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.
1968/71 -1982	Ministry of Works Code of Practice: 1968	Ministry of Works Code of Practice: 1968	Ultimate Limit State (ULS)/Limit State Design (LSD) recommended. Detailing requirements introduced for (i) beam-column joints; (ii) column confinement. **Capacity design** introduced between beams and columns (though no allowance for beam overstrength due to slab reinforcement contribution).

Period	Loading Standard	Concrete Standard	Major changes
	NZ4203:1976	ACI 318:1971 or provisional NZ Concrete Standard, NZS 3101:1970	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). Capacity design required to ensure sum of column strengths greater than the sum of beam strengths (with no minimum ratio).
1982-1995	NZS 4203:1984	NZS 3101:1982	Modifications to strength reduction factors: 0.9 for flexure in beams and confined columns; 0.7 for unconfined column with axial load higher than $0.1A_g f'_c$; 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) **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 ϕ=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).

Period	Loading Standard	Concrete Standard	Major changes
			• 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 overstrength 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.
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.

Period	Loading Standard	Concrete Standard	Major changes
			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 $(N^*/(A_g f'_c))$ 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)
2006-	NZS 1170.5: 2004	NZD3101:2006	**Building classifications** 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).

Period	Loading Standard	Concrete Standard	Major changes
			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, μ, 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). **Materials** 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 overstrength 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 overstrength 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.

Period	Loading Standard	Concrete Standard	Major changes
			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 overstrength conditions than in normal ultimate strength design conditions. As strain levels increase the width of floor 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 overstrength 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 ULS 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 (SLS) 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 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 SLS 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 SLS 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 SLS 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.

Period	Loading Standard	Concrete Standard	Major changes
			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) – 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) – specifying requirements for shear strength of precast units in zones where overstrength 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) – 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 overstrength actions act at the supports and vertical ground motion induces negative moments in the floor (clause 19.4.3.6) – 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).

C5A.3 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 C5Esummarises the structural detail requirements for beams according to the NZS 3101:2006 standards from 1970 onwards (1970, 1982, 1995 and 2006).

Figure C5A.1 illustrates the evolution of structural design requirements and detailing layout for beams according to the New Zealand concrete standard from the 1970s onwards.

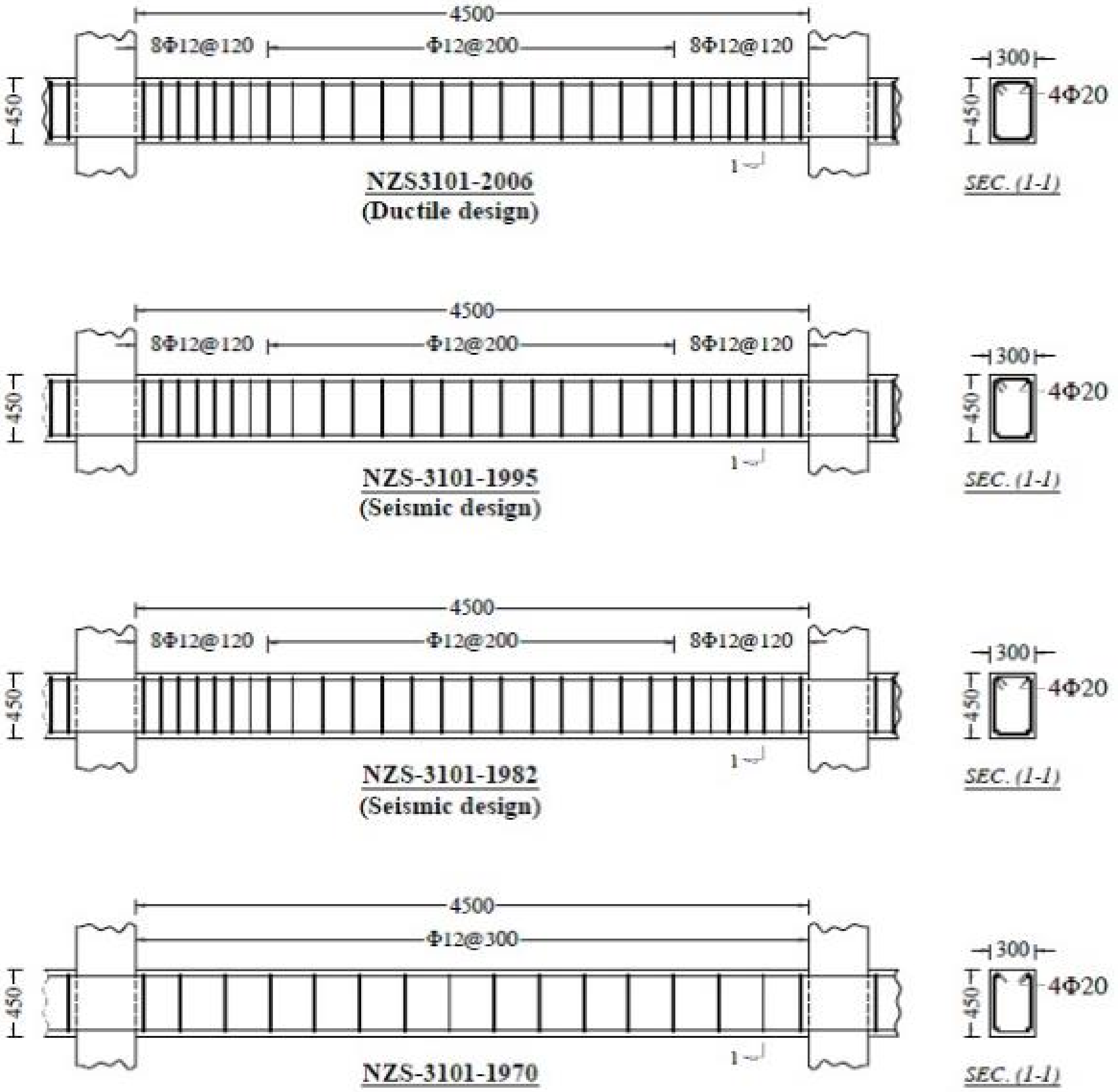


Figure C5A.1: Example of typical beam layouts according to different versions of NZS 3101:2006 (Cuevas et al., 2015)

C5A.4 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 C5E 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 C5A.2 and C5A.3 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.

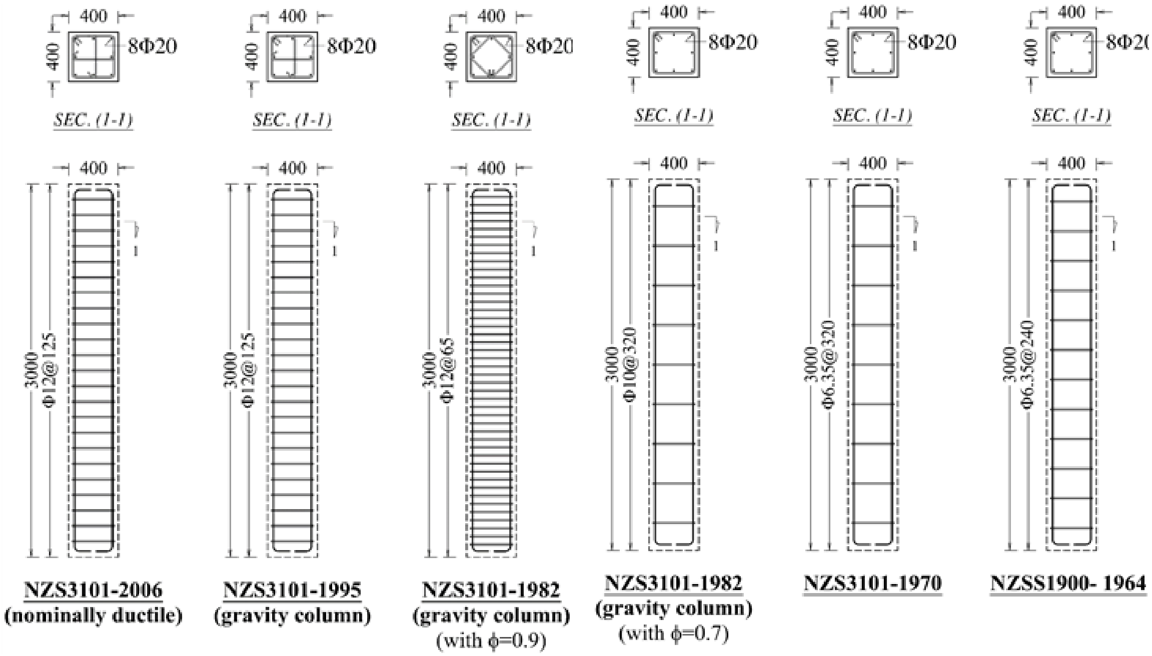


Figure C5A.2: Example of typical gravity column layouts according to different New Zealand concrete standards from the mid-1960s on (Niroomandi et al., 2015)

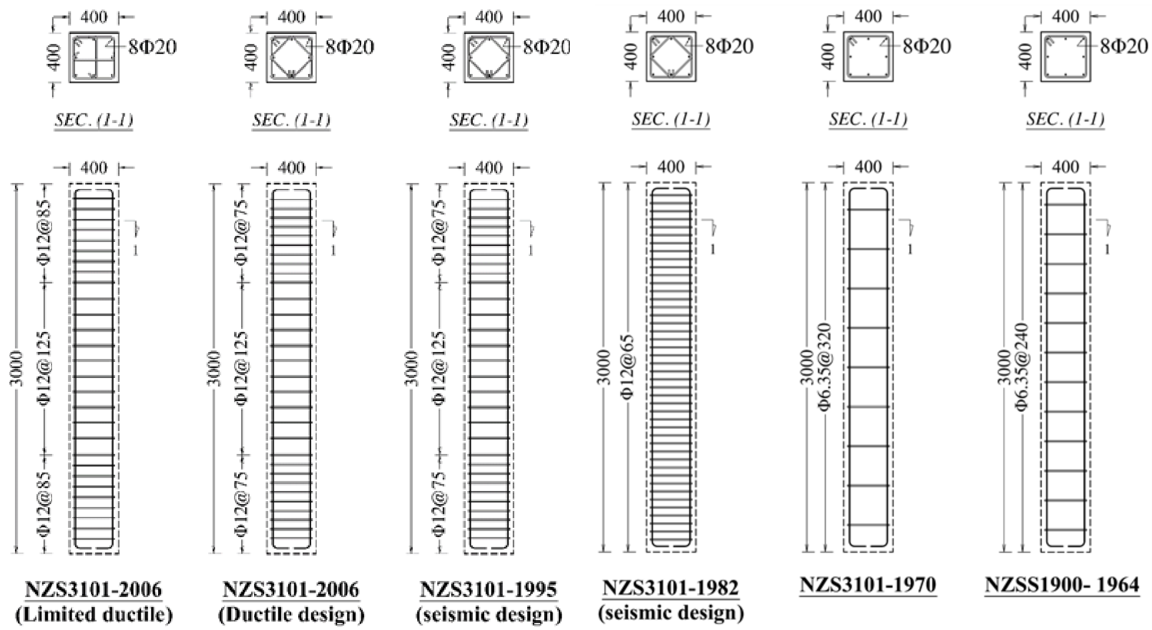


Figure C5A.3: Example of typical column layouts with seismic design according to different New Zealand concrete standards from the mid-1960s onwards (Niroomandi et al., 2015)

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, $\upsilon\Delta$, 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.

The following table 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 C5E (Niromaandi et al., 2015).

Table C5A.2: Comparison of transverse reinforcement spacing requirements in concrete structures standards (Stirrat et al., 2014)

Design standard	Non-seismic spacing limit	Limited-ductile spacing limit	Ductile spacing limit
NZS 1900 Chapter 9.3:1964	For spirally-wound columns, min. of 75 mm or $d_c/6$		
NZS 3101:1982	Min. of h, b_c, $16d_b$, $48d_{bt}$	Min. of h, b_c, $10d_b$, $48d_{bt}$	Min. of $h/5$, $b_c/5$, $6d_b$, 200 mm
NZS 3101:1995 and NZS 3101:2006	Min. of $h/3$, $b_c/3$, $10d_b$	Min. of $h/4$, $b_c/4$, $10d_b$	Min. of $h/4$, $b_c/4$, $6d_b$

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.

C5A.5 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 C5E summarises the minimum design requirements for beam-column joint reinforcement and details according to NZS 3101:1970, 1982, 1995 and 2006.

Figure C5A.4 illustrates the evolution of structural design requirements and detailing layout for beams according to these standards.

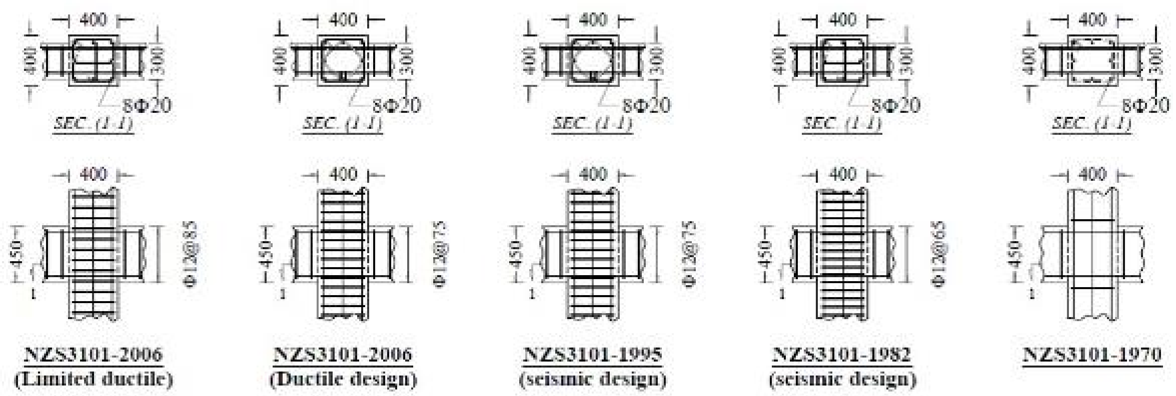
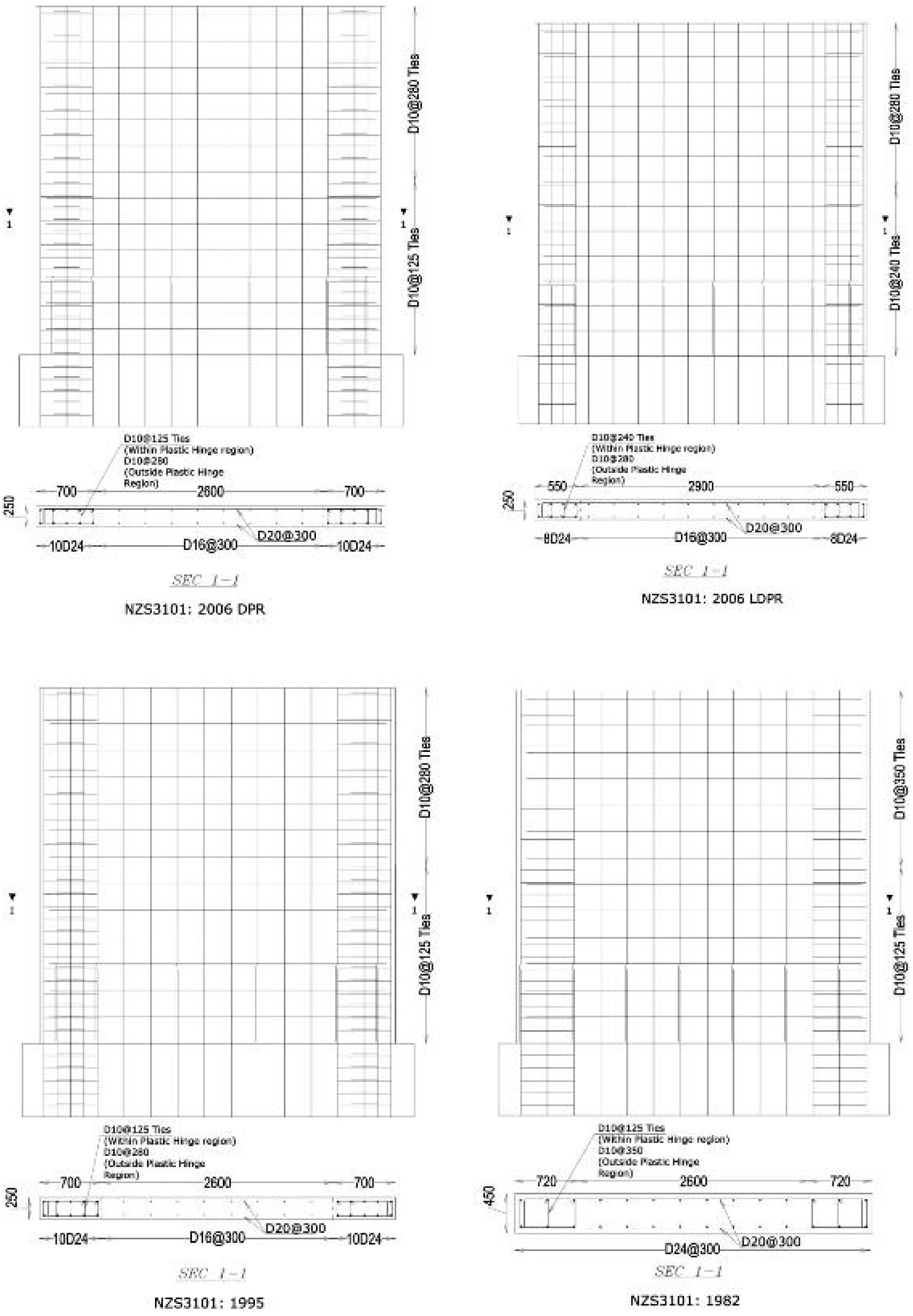


Figure C5A.4: Example of typical beam-column joint layouts according to different New Zealand standards (Cuevas et al., 2015)

C5A.6 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 C5E 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.

The following figure illustrates an example of the evolution of structural design requirements and detailing layout for shear walls according to these standards.



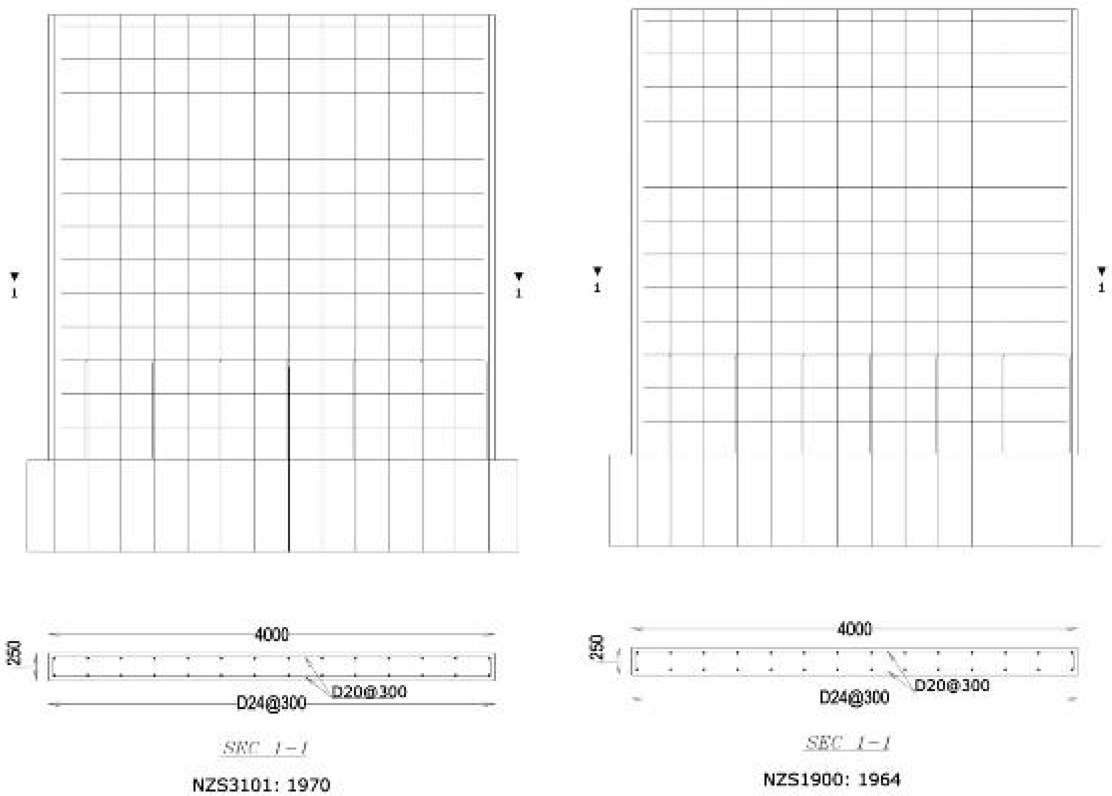


Figure C5A.5: Example of typical reinforcement layouts for shear walls designed according to different New Zealand concrete standards from mid-1960s on (Dashti et al., 2015)

Appendix C5B Historical Concrete Property Requirements, Design Specifications and Requirements for Concrete Strength Testing in New Zealand

This appendix provides tables comparing the:

- concrete property requirements and design specifications from NZS 3101:1970 to NZS 3010:2006, and
- concrete strength tests for quality control from (NZS 3104:1983 to NZS 3104:2003.

Table C5B.1: Comparison of concrete property requirements and design specifications from four generations of New Zealand standards post-1970

Standard / Concrete Property	(1) NZS 3101:1970	NZS 3101:1982	NZS 3101: 1995	NZS 3101: 2006
Specified compressive strength (MPa)	f'_c = 17.2 MPa, 20.7 MPa, 27.6 MPa, 34.5 MPa	20 MPa < f'_c < 55 MPa	17.5 MPa < f'_c < 100 MPa	25 MPa ≤ f'_c < 100 MPa 25 MPa ≤ f'_c < 75 MPa (for ductile elements and elements of limited ductility)
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: $f_r=0.8\times1.8\,f_{ct}$ (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-light-weight concrete) 0.85 (for sand-light-weight concrete)	• $f_r=0.6\,\lambda\sqrt{f'_c}$ (for the purpose of calculation deflections) λ = 0.85 (normal weight sand, lightweight coarse aggregate) λ = 0.75 (lightweight sand, lightweight coarse aggregate) λ = 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)				($0.36\sqrt{f'_c}$) or (0.54 × indirect tensile strength obtained from Brazil test according to AS 1012:Part 10)
Elastic Modulus	E = 0.043 $w^{1.5}\sqrt{f'_c}$ (for 1450 < w(2) < 2500 kg/m^3)	E = 0.043 $w^{1.5}$ $\sqrt{f'_c}$ (for 1400 < w < 2500 kg/m^3) E=4700 $\sqrt{f'_c}$ (for normal weight concrete)	E = (3320 $\sqrt{f'_c}$+6900)$\left(\frac{\rho}{2300}\right)^{1.5}$ (for 1400 < ρ < 2500 kg/m^3) E=(3320 $\sqrt{f'_c}$+6900) (for normal weight concrete)	Testing of plain concrete E = (3320$\sqrt{f'_c}$+6900)$\left(\frac{\rho}{2300}\right)^{1.5}$ (for 1400 < ρ < 2500 kg/m^3) E = (3320 $\sqrt{f'_c}$+6900) (for normal weight concrete) E ≥ value corresponding to (f'_c+10) MPa (when strain induced action are critical) Note: For the SLS, this value may be used in lieu of above expression.
Poisson ratio			0.2	0.2 (for normal density concrete) Shall be determined (for lightweight concrete)
Coefficient of thermal expansion (/°C)			12 × 10^{-6}	For concrete of an aggregate type: • Greywacke (9.5 -11 × 10^{-6}) • Phonolite (10.0 -11.0 × 10^{-6}) • Basalt (9.0 -10.0 × 10^{-6}) • Andesite (7.0 – 9.0 × 10^{-6}) The coefficient of thermal expansion may be taken as 12 × 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.

Table C5B.2: Concrete strength tests for quality control

Standard / Control Tests	NZS 3104:1983	NZS 3104:1991	NZS 3104:2003
Number of test specimens	3 specimens made from one sample of concrete 2 specimens when the number of tests > 20 and the 28-day compressive testing mean has a within-test coefficient of variation of the test series of less than 4%.	Same as NZS 3104:1983	Same as NZS 3104:1983
Frequency of testing	• Ready-mixed concrete: – 1/75 m^3 (up to 15,000 m^3 per annum), with an additional test for every 250 m^3 above 15,000 m^3 – At least 120 tests per annum • Site-mixed concrete: – 1 sample (each day/75 m^3)	Same as NZS 3104:1983	• Ready-mixed concrete: – Same as NZS 3104:1983 – At least 10 tests per month (6 tests per month in the case of plants producing less than 9000 m^3 per annum) • Site-mixed concrete: – Same as NZS 3104:1983

Appendix C5C Historical Reinforcing Steel Properties in New Zealand

C5C.1 General

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 of structural 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:

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 (Presland, 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.

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 C5C.1 summarises the evolution of these standards, while Tables C5C.2 to C5C.4 in the next section list available diameters for steel reinforcing bars. Also refer to Appendix C5E for a summary of the historical evolution of the mechanical properties of steel reinforcing over different time periods.

Table C5C.1: Evolution of reinforcing steel material standards in New Zealand

<table>
<tr><th>1949</th><th>1962</th><th>1964</th><th>1968</th><th>1972</th><th>1973</th><th>1989</th><th>2001</th></tr>
<tr><td colspan="4">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 Table C5D.2)</td><td>NZS 3423P:1972
Hot rolled plain round steel bars of structural grade for reinforced concrete
“Grade” 40,000 psi
(275 MPa)</td><td rowspan="3">NZS 3402P:
1973
Hot rolled steel bars for the reinforcement of concrete
Grade
275 MPa
Grade
380 MPa</td><td rowspan="3">NZS 3402:
1989
Steel bars for the reinforce-ment of concrete
Grade
300 MPa
Grade
430 MPa</td><td rowspan="3">AS/NZS 4671:
2001
Steel reinforcing material
Grade
300 MPa
Grade
500 MPa</td></tr>
<tr><td></td><td>NZSS 1693:1962
Deformed steel bars of structural grade for reinforced concrete
“Grade” 33000 psi
(227 MPa)</td><td colspan="3">NZSS 1693:1962
(Amendment 1:1968)
Deformed steel bars of structural grade for reinforced concrete
“Grade” 40000 psi
(275 MPa)</td></tr>
<tr><td></td><td></td><td colspan="3">NZS 1879:1964
Hot rolled deformed bars of HY 60
(High Yield 60,000 psi) for reinforced concrete
Grade” 60,000 psi (415 MPa)</td></tr>
</table>

C5C.2 Mechanical Properties of Steel Reinforcing Bars Over Different time Periods

The evolution of standards for the mechanical properties of steel reinforcement bars is summarised in the following tables.

Table C5C.2: Mechanical properties of steel reinforcement bars – pre-1960s

Steel Property / **Standard**	**NZS 197:1949 (BS 785:1938)**			
Type of steel	Plain round bar *Mild steel (MS)* *Medium tensile (MT)* *High tensile (HT)*			
Yielding stress	*Bar size (diameter)*	*MS*	*MT*	*HT*
	Up to 1 inch	Not Specified	19.5 tsi (≈270 MPa)	23.0 tsi (≈317 MPa)
	Over 1 to 1½ inch		18.5 tsi (≈255 MPa)	22.0 tsi (≈303 MPa)
	Over 1½ to 2 inch		17.5 tsi (≈241 MPa)	21.0 tsi (≈290 MPa)
	Over 2 to 2½ inch		16.5 tsi (≈227 MPa)	20.0 tsi (≈275 MPa)
	Over 2½ to 3 inch		16.5 tsi (≈227 MPa)	19.0 tsi (≈262 MPa)
Tensile strength		≥ 28 tsi (≈ 386 MPa)	≥ 33 tsi (≈ 455 MPa)	≥ 37 tsi (≈ 510 MPa)
		≤ 33 tsi (≈ 455 MPa)	≤ 38 tsi (≈ 524 MPa)	≤ 43 tsi (≈ 593 MPa)
Elongation at fracture (%)	Up to 1 inch	≥ 20[1]	≥ 18[1]	≥ 18[1]
	Over 1 to 1½ inch	≥ 16[1]	≥ 14[1]	≥ 14[1]
	Under ⅜ inch	≥ 24[2]	≥ 22[2]	≥ 22[2]

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 C5C.3: Mechanical properties of steel reinforcement bars – 1960s to mid-1970s

Steel Property / **Standard**	**NZS 197:1949 (BS 785:1938)**			
Type of steel	Plain round bar *Mild steel (MS)* *Medium tensile (MT)* *High tensile (HT)*			
Yielding stress	*Bar size (diameter)*	*MS*	*MT*	*HT*
	Up to 1 inch	Not Specified	19.5 tsi (≈270 MPa)	23.0 tsi (≈317 MPa)
	Over 1 to 1½ inch		18.5 tsi (≈255 MPa)	22.0 tsi (≈303 MPa)
	Over 1½ to 2 inch		17.5 tsi (≈241 MPa)	21.0 tsi (≈290 MPa)
	Over 2 to 2½ inch		16.5 tsi (≈227 MPa)	20.0 tsi (≈275 MPa)
	Over 2½ to 3 inch		16.5 tsi (≈227 MPa)	19.0 tsi (≈262 MPa)
Tensile strength		≥ 28 tsi (≈ 386 MPa)	≥ 33 tsi (≈ 455 MPa)	≥ 37 tsi (≈ 510 MPa)
		≤ 33 tsi (≈ 455 MPa)	≤ 38 tsi (≈ 524 MPa)	≤ 43 tsi (≈ 593 MPa)
Elongation at fracture (%)	Up to 1 inch	≥ 20[(1)]	≥ 18[(1)]	≥ 18[(1)]
	Over 1 to 1½ inch	≥ 16[(1)]	≥ 14[(1)]	≥ 14[(1)]
	Under ⅜ inch	≥ 24[(2)]	≥ 22[(2)]	≥ 22[(2)]

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 C5C.4: Mechanical properties of steel reinforcement bars – 1970s onwards

Steel Property \ Standard	NZ 3402P:1973		NZS 3402:1989		AS/NZS 4671:2001	
Type of steel	Grade 275	Grade 380	Grade 300	Grade 430	Grade 300	Grade 500
Yielding stress (MPa) • Lower bound • Upper bound	275	380	≥ 275$^{(min)}$ (300$^{(k)}$) ≤ 380$^{(max)}$ (355$^{(k)}$)	≥ 410$^{(min)}$ (430$^{(k)}$) ≤ 520$^{(max)}$ (500$^{(k)}$)	≥ 300$^{(k)}$ ≤ 380$^{(k)}$	≥ 500$^{(k)}$ ≤ 600$^{(k)}$
Tensile Strength (MPa)	≥ 380 ≤ 520	≥ 570*	Not specified		Not specified	
Ratio $R_{\mathrm{m}}/R_{\mathrm{e}}$ (TS/YS)	Not specified		$1.15 \le \frac{TS}{YS} \le 1.50$	$1.15 \le \frac{TS}{YS} \le 1.40$	$1.15 \le \frac{R_{\mathrm{m}}}{R_{\mathrm{e}}} \le 1.50$	$1.15 \le \frac{R_{\mathrm{m}}}{R_{\mathrm{e}}} \le 1.40$
Elongation at maximum force A_{gt} (%)	Not specified		Not specified		≥ 15	≥ 10
Elongation at fracture (%)	≥ 20[1]	≥ 12[1]	≥ 20[1]	≥ 12[1]	Not specified	

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)*

C5C.3 Mechanical Properties of Mesh

The evolution of Standards for hard drawn steel wire and mesh for concrete reinforcement is shown in Table C5C.5.

Table C5C.5: Evolution of hard drawn steel wire and mesh for concrete reinforcement standards in New Zealand

1949	1972	1975	2001
NZS 197:1949 (BS 785:1938) Rolled steel bars and hard drawn steel wire for concrete reinforcement	NZS 3421:1972 Hard drawn steel wire for concrete reinforcement (metric and imperial units)	NZS 3421:1975 Hard drawn steel wire for concrete reinforcement (metric units)	AS/NZS 4671:2001 Steel reinforcing material
	NZS 3422:1972 Welded fabric of drawn steel wire for concrete reinforcement (metric units)	NZS 3422:1975 Welded fabric of drawn steel wire for concrete reinforcement (metric units)	

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/in^2 (510 MPa) and 42 ton/in^2 (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/in^2 (483 MPa) while the minimum tensile strength was 83,000 lbf/in^2 (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/in^2 (855 MPa) for diameters up to and including 0.128 in (3.25 mm) and 112,000 lbf/in^2 (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.

Appendix C5D Test Methods for Investigating Material Properties

C5D.1 Concrete

The following table summarises test methods for investigating concrete material properties.

Table C5D.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)

Method	Capability/Use	Advantages	Disadvantages
DESTRUCTIVE TESTS			
Compressive test	Strength of concrete	Direct evaluation of concrete strength from compressive tests on cylindrical specimens	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
SEMI-DESTRUCTIVE TESTS			
Pull-out	In-place estimation of the compressive and tensile strengths	In-place strength of concrete can be quickly measured	Pull-out device must be inserted in a hole drilled in the hardened concrete Only a limited depth of material can be tested
Pull-off/tear-off	Direct tension test	In situ tensile strength of concrete Determining bond strength between existing concrete and repair material	Sensitivity to rate of loading
Penetration probe (Windsor probe)	Estimation of compressive strength, uniformity and quality of concrete Measuring the relative rate of strength development of concrete at early ages	The equipment is easy to use (not requiring surface preparation) The results are not subject to surface conditions and moisture content	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
NON-DESTRUCTIVE TESTS			
Visual tests	The first step in investigating concrete material	Quick evaluation of damage	No detailed information
Rebound hammer	Measuring surface hardness of concrete to estimate compressive strength	The assessment of the surface layer strength	Results can only suggest the hardness of surface layer
Durability test: Concrete electrical resistivity	Measuring the ability of the concrete to conduct the corrosion current	Inexpensive, simple and many measurements can be made rapidly	Not reliable at high moisture content

Method		Capability/Use	Advantages	Disadvantages
	Permeability	To evaluate the transfer properties of concrete (porosity)	Useful method to evaluate the risk of leaching, corrosion and freezing	Thickness limitation Age, temperature dependent Sufficient lateral sealing
Fiberscope		To check the condition of cavities, and honeycombing in reinforced concrete Voids detection along grouted post-stressed tendons	Direct visual inspection of inaccessible parts of an element	Semi destructive as the probe holes usually must be drilled Needs additional fibre to carry light from an external source
Stress-Wave propagation methods	Ultrasonic pulse velocity	Evaluation of concrete strength and quality Identification of internal damage and location of reinforcement	Excellent for determining the quality and uniformity of concrete; especially for rapid survey of large areas and thick members	The measure can be distorted by the presence of lesions in the concrete The test requires smooth surfaces for a good adhesion of the probes No information about the depth of suspected flaw
	Ultrasonic echo method	Quality control and integrity of concrete	Access to only one face is needed Internal discontinuities and their sizes can be estimated	Limited member thickness
	Impact echo method	Defects within concrete element such as delamination, voids, honeycombing	Access to only one face is needed	The ability of instrument is limited to less than 2 m thickness
	Spectral analysis of surface waves	Determining the stiffness profile of a pavement Depth of deteriorated concrete	Capability of determining the elastic properties of layered systems such as pavement and interlayered concrete	Complex signal processing
Nuclear methods	Gamma radiography	Location of internal cracks, voids and variations in density of concrete	Simple to operate Applicable to a variety of materials	X-ray equipment is bulky and expensive Difficult to identify cracks perpendicular to radiation beam
	Backscatter radiometry	Determining in-place density of fresh or hardened concrete	Access only to surface of test object Since this method's measurements are affected by the top 40 to 100 mm, best for assessing surface zone of concrete element	The accuracy of this method is lower than direct transmission Measurements are influenced by near surface material and are sensitive to chemical composition
	CT scanning	Concrete imaging	3D crack/damage monitoring	Sophisticated software for analysis Not in situ application Access to CT scanner needed

Method	Capability/Use	Advantages	Disadvantages
Infrared thermography	Detecting delamination, heat loss and moisture movement through concrete elements; especially for flat surfaces	Permanent records can be made Tests can be done without direct access to surface by means of infrared cameras	Expensive technique Reference standards are needed Very sensitive to thermal interference from other heat sources The depth and thickness of subsurface anomaly cannot be measured
Ground penetrating radar	Identification of location of reinforcement, depth of cover, location of voids and cracks Determination of in situ density and moisture content	Can survey large areas rapidly	Results must be correlated to test results on samples obtained Low level signals from targets as depth increases
Acoustic emission	Real time monitoring of concrete degradation growth and structural performance	A few transducers are enough to locate defects over large areas Can detect the initiation and growth of cracks in concrete under stress	Passive technique, could be used when the structure is under loading
Ultrasonic tomography (MIRA)	Uses high frequency (greater than 20,000 Hz) sound waves to characterise the properties of materials or detect their defects	Thickness measurement, reinforcement location, and distress evaluation	Significant efforts and user expertise are required for measurement and data interpretation of large scale application
Petrography	Forensic investigation of concrete Determining the composition and identifying the source of the materials Determination of w/c Determining the depth of fire damage	Microscopic examination of concrete samples	Laboratory facilities as well as highly experienced personnel are needed to interpret the result
Sclerometric method	Determination of compressive strength	Determination of a sclerometric index connected to compressive strength	The instrument must be in the horizontal direction or the reliability of results is reduced Empirical formulas, based on probabilistic methods, are used to obtain the concrete strength The preparation of the test surface is laborious and expensive
SonReb method	Determination of compressive strength	The concomitant use of sclerometric and ultrasonic methods can reduce mistakes due to the influence of humidity and aging of concrete	Risk of regression on a small statistically representative sample

C5D.2 Reinforcing Steel

The following table summarises test methods for investigating reinforcing steel material properties.

Table C5D.2: Destructive and non-destructive tests for investigating reinforcing steel material properties (De Pra, Bianchi and Pampanin, 2015)

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

Appendix C5E Evolution of Standard Based Design Details for Reinforcement and Detailing

C5E.1 Beams

The following table summarises the evolution of standard-based details requirements for beams, from NZS 3101P:1970 to NZS 3101:2016.

Table C5E.1: Evolution of standard-based details requirements for beams

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970
Lateral support spacing	$50b_{\mathrm{w}}$	$50b_{\mathrm{w}}$ (for earthquake)	$50b_{\mathrm{w}}$	
$\rho_{\max}$	$\rho_{\max} = 0.75\rho_{\mathrm{bal}}$	$\rho_{\max} = 0.75\rho_{\mathrm{bal}}$	$\rho_{\max} = 0.75\rho_{\mathrm{bal}}$	$\rho_{\max} = 0.75\rho_{\mathrm{bal}}$ (for USD)
$\rho_{\min}$	$\rho_{\min} = \frac{\sqrt{f'_{\mathrm{c}}}}{4f_{\mathrm{y}}} \geq \frac{1.4}{f_{\mathrm{y}}}$	$\rho_{\min} = \frac{\sqrt{f'_{\mathrm{c}}}}{4f_{\mathrm{y}}} \geq \frac{1.4}{f_{\mathrm{y}}}$	$\rho_{\min} = \frac{1.4}{f_{\mathrm{y}}}$	$\rho_{\min} = \frac{200}{f_{\mathrm{y}}}$
$\rho_{\min}$ (alternatively)	$\rho_{\min} = \frac{4}{3}\rho_{\mathrm{reqd}}$ (for gravity only)	$\rho_{\min} = \frac{4}{3}\rho_{\mathrm{reqd}}$	$\rho_{\min} = \frac{4}{3}\rho_{\mathrm{reqd}}$	$\rho_{\min} = \frac{4}{3}\rho_{\mathrm{reqd}}$
Maximum d_{b} in internal beam-column joints (for nominally ductile structures)	$\frac{d_{\mathrm{b}}}{h_{\mathrm{c}}} = 4\alpha_f \frac{\sqrt{f'_{\mathrm{c}}}}{f_{\mathrm{y}}}$ $\alpha_{\mathrm{f}} = 0.85$ (two-way) $\alpha_{\mathrm{f}} = 1.00$ (one-way)			
Minimum requirements for transverse reinforcement	5 mm in diameter $f_{\mathrm{yt}} \leq 500MPa$		6 mm in diameter (for earthquake)	
Maximum nominal shear stress	$v_{\mathrm{n}} \leq 0.2f'_{\mathrm{c}}$ or $8MPa$	$v_{\mathrm{n}} \leq \begin{cases} 0.2f'_{\mathrm{c}} \\ 1.1\sqrt{f'_{\mathrm{c}}} \\ 9MPa \end{cases}$	$v_{\mathrm{n}} \leq 0.2f'_{\mathrm{c}}$ or $6MPa$	$v_{\mathrm{n}} \leq 5\sqrt{f'_{\mathrm{c}}}$ $v_{\mathrm{n}} \leq 8.5\sqrt{f'_{\mathrm{c}}}$ (USD)
Spacing limits for shear reinforcement	$S_{\max} \leq \begin{cases} 0.5d \;\;, b_{\mathrm{w}} \\ 500\ mm \\ 16d_{\mathrm{b}} \end{cases}$	$S_{\max} \leq \begin{cases} 0.5d \\ 600\ mm \end{cases}$	$S_{\max} \leq \begin{cases} 0.5d \\ 600\ mm \end{cases}$	$S_{\max} \leq \begin{cases} 0.75d \\ 450\ mm \end{cases}$
	($0.5d$ and $500\ mm$ reduced by half if $v_{\mathrm{s}} \geq 0.33\sqrt{f'_{\mathrm{c}}}$)	($0.5d$ and $500\ mm$ reduced by half if $v_{\mathrm{s}} \geq 0.07f'_{\mathrm{c}}$)	($0.5d$ and $500\ mm$ reduced by half if $v_{\mathrm{s}} \geq 0.07f'_{\mathrm{c}}$)	($S_{\max} \leq 0.25d$ if $v_{\mathrm{n}} \geq 3\sqrt{f'_{\mathrm{c}}}$, or $v_{\mathrm{n}} \geq 5.1\sqrt{f'_{\mathrm{c}}}$ for USD)
Minimum area of shear reinforcement	$A_{\mathrm{v}} = \frac{1}{16}\sqrt{f'_{\mathrm{c}}}\frac{b_{\mathrm{w}}s}{f_{\mathrm{yt}}}$	$A_{\mathrm{v}} = 0.35\frac{b_{\mathrm{w}}s}{f_{\mathrm{yt}}}$	$A_{\mathrm{v}} = 0.35\frac{b_{\mathrm{w}}s}{f_{\mathrm{yt}}}$	$A_{\mathrm{v}} = 0.0015b_{\mathrm{w}}s$
Dimension of beams (for earthquake)	$\frac{L_{\mathrm{n}}}{b_{\mathrm{w}}} \leq 25$ $\frac{L_{\mathrm{n}}h}{b_{\mathrm{w}}^2} \leq 100$ $b_{\mathrm{w}} \geq 200\ mm$	$\frac{L_{\mathrm{n}}}{b_{\mathrm{w}}} \leq 25$ $\frac{L_{\mathrm{n}}h}{b_{\mathrm{w}}^2} \leq 100$ $b_{\mathrm{w}} \geq 200\ mm$	$\frac{L_{\mathrm{n}}}{b_{\mathrm{w}}} \leq 25$ $\frac{L_{\mathrm{n}}h}{b_{\mathrm{w}}^2} \leq 100$ $b_{\mathrm{w}} \geq 200\ mm$	

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970
ρ_{max} (for earthquake, within plastic hinge region)	$\rho_{\mathrm{max}} = \frac{f'_{\mathrm{c}} + 10}{6 f_{\mathrm{y}}}$ ≤ 0.025	$\rho_{\mathrm{max}} = \frac{f'_{\mathrm{c}} + 10}{6 f_{\mathrm{y}}}$ ≤ 0.025	$\rho_{\mathrm{max}} = \frac{1 + 0.17\left(\frac{f'_{\mathrm{c}}}{7} - 3\right)}{100} \left(1 + \frac{\rho'}{\rho}\right) \leq \frac{7}{f_{\mathrm{y}}}$	
ρ_{min} (for earthquake, within plastic hinge region)	$A'_{s} > 0.5 A_{\mathrm{s}}$ for ductile plastic regions. $A'_{s} > 0.38 A_{\mathrm{s}}$ for limited ductile plastic regions. $\rho_{\mathrm{min}} = \frac{\sqrt{f'_{\mathrm{c}}}}{4 f_{\mathrm{y}}}$	$A'_{\mathrm{s}} > 0.5 A_{\mathrm{s}}$ $\rho_{\mathrm{min}} = \frac{\sqrt{f'_{\mathrm{c}}}}{4 f_{\mathrm{y}}}$	$A'_{\mathrm{s}} > 0.5 A_{\mathrm{s}}$ $\rho_{\mathrm{min}} = \frac{1.4}{f_{\mathrm{y}}}$	
Maximum longitudinal beam bar diameter to column depth (for earthquake)	$\frac{d_{\mathrm{b}}}{h_{\mathrm{c}}} \leq 3.3 \alpha_{\mathrm{f}} \alpha_{\mathrm{d}} \frac{\sqrt{f'_{\mathrm{c}}}}{1.25 f_{\mathrm{y}}}$ $f'_{\mathrm{c}} \leq 70 MPa$ $\alpha_{\mathrm{d}} = 1.00$ (ductile) $\alpha_{\mathrm{d}} = 1.20$ (limited ductile)			
Minimum area of shear reinforcement (for earthquake)	$A_{\mathrm{v}} = \frac{1}{12} \sqrt{f'_{\mathrm{c}}} \frac{b_{\mathrm{w}} s}{f_{\mathrm{yt}}}$			
Spacing limits for shear reinforcement (for earthquake)	$S_{\mathrm{max}} = \begin{cases} 12 d_{\mathrm{b}} \\ d/2 \end{cases}$	$S_{\mathrm{max}} = \begin{cases} 16 d_{\mathrm{b}} \\ b_{\mathrm{w}} \end{cases}$	$S_{\mathrm{max}} \leq \begin{cases} b_{\mathrm{w}} \\ 48 d_{\mathrm{v}} \\ 16 d_{\mathrm{b}} \end{cases}$	
Minimum area of shear reinforcement in plastic hinge regions (for earthquake)	$A_{\mathrm{te}} = \frac{\sum A_{\mathrm{b}} f_{\mathrm{y}}}{96 f_{\mathrm{yt}}} \frac{s}{d_{\mathrm{b}}}$	$A_{\mathrm{te}} = \frac{\sum A_{\mathrm{b}} f_{\mathrm{y}}}{96 f_{\mathrm{yt}}} \frac{s}{d_{\mathrm{b}}}$	$A_{\mathrm{te}} = \frac{\sum A_{\mathrm{b}} f_{\mathrm{y}}}{160 f_{\mathrm{yt}}} \frac{s}{d_{\mathrm{b}}}$	
Spacing limits for shear reinforcement in plastic hinge regions (for earthquake)	$S_{\mathrm{max}} = \begin{cases} 6 d_{\mathrm{b}} \\ d/4 \end{cases}$ (ductile) $S_{\mathrm{max}} = \begin{cases} 10 d_{\mathrm{b}} \\ d/4 \end{cases}$ (limited ductile)	$S_{\mathrm{max}} = \begin{cases} 6 d_{\mathrm{b}} \\ d/4 \end{cases}$	$S_{\mathrm{max}} = \begin{cases} 6 d_{\mathrm{b}} \\ d/4 \end{cases}$	
Maximum nominal shear stress (for earthquake)		$v_{\mathrm{n}} \leq \begin{cases} 0.16 f'_{\mathrm{c}} \\ 0.85 \sqrt{f'_{\mathrm{c}}} \end{cases}$		

Note:

NZS 3101P:1970 units of [psi]

USD: Ultimate Strength Design

C5E.2 Columns

The following table summarises the evolution of standards-based details requirements for columns, from NZSS 1990 (1964) to NZS 3101:2006.

Table C5E.2: Evolution of standards-based details requirements for columns (Niroomandi et al., 2015)

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
Strength reduction factor (ϕ)	0.85	0.85	0.9 for conforming transverse 0.7 for others	0.75 for spirally reinforced 0.7 for tied	-
f_c'	$25 - 100\ MPa$ For DPRs[5] and LDPRs[6]: $25 - 70\ MPa$	-	-	-	-
f_y	$< 500\ MPa$	-	-	-	-
f_{yt}	$< 500\ MPa$ for shear $< 800\ MPa$ for confinement	$< 500\ MPa$ for shear $< 800\ MPa$ for confinement	$< 400\ MPa$	$< 414\ MPa$	-
Maximum axial compressive load	$0.85\phi N_{\mathrm{n,max}}$[1] For DPRs and LDPRs: $0.7\phi N_{\mathrm{n,max}}$[1]	$0.85\phi N_{\mathrm{n,max}}$[1] For DPRs: $0.7\phi N_{\mathrm{n,max}}$[1]	$0.85\phi N_{\mathrm{n,max}}$[1] for conforming, otherwise $0.8\phi N_{\mathrm{n,max}}$[1] For DPRs: Min of ($0.7\phi f_c' A_{\mathrm{g}}$ and $0.7\phi N_{\mathrm{n,max}}$[1])	P_0[18]	For tied columns: $P = cA_{\mathrm{c}} + ncA$ For spirally columns: $P = cA_{\mathrm{k}} = ncA + 2t_{\mathrm{b}}A_{\mathrm{b}}$
Dimension of column	For DPRs and LDPRs: $b_w \geq L_{\mathrm{n}}/25$ $b_{\mathrm{w}} \geq \sqrt{L_{\mathrm{n}}h/100}$	For DPRs: $b_w \geq L_{\mathrm{n}}/25$ $b_w \geq \sqrt{L_{\mathrm{n}}h/100}$	For DPRs: $\mathrm{b_w} \geq L_{\mathrm{n}}/25$ $b_w \geq \sqrt{L_{\mathrm{n}}h/100}$	25.4 mm for circular 20.32 for rectangular or $A_g >$ 413 mm^2	-
Extend of ductile detailing length, l_y, for detailing purposes	For DPRs and LDPRs: $l_y = h$ for $N_0^* \leq 0.25\phi f_c' A_{\mathrm{g}}$ $l_y = 2h$ for $0.25\phi f_c' A_{\mathrm{g}} < N_0^* \leq 0.5\phi f_c' A_{\mathrm{g}}$ $l_y = 3h$ for N_0^*[2] $> 0.5\phi f_c' A_{\mathrm{g}}$	For DPRs: $l_y = h$ for $N_0^* \leq 0.25\phi f_c' A_{\mathrm{g}}$ $l_y = 2h$ for $0.25\phi f_c' A_{\mathrm{g}} < N_0^* \leq 0.5\phi f_c' A_{\mathrm{g}}$ $l_y = 3h$ for N_0^*[2] $> 0.5\phi f_c' A_{\mathrm{g}}$	For DPRs: $l_y = h$ for P_{e}[13] $\leq 0.3\phi f_c' A_{\mathrm{g}}$ $l_y = 1.5h$ for $P_{\mathrm{e}} > 0.3\phi f_c' A_{\mathrm{g}}$	-	-
Minimum longitudinal reinforcement ratio	$0.008A_{\mathrm{g}}$	$0.008A_{\mathrm{g}}$	$0.008A_{\mathrm{g}}$	0.01	0.008

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
Maximum longitudinal reinforcement ratio	$0.08A_g$ For DPRs and LDPRs: $18\,A_g/f_y$	$0.08A_g$ For DPRs: $18\,A_g/f_y$	$0.08A_g$ For DPRs: $0.06A_g$ for Grade 275 $0.045A_g$ for Grade 380	0.08	0.08
Maximum longitudinal reinforcement ratio at splices	$0.08A_g$ For DPRs and LDPRs: $24\,A_g/f_y$	$0.08A_g$ For DPRs: $24\,A_g/f_y$	For DPRs: $0.08A_g$ for Grade 275 $0.06A_g$ for Grade 380	0.12	-
Minimum number of longitudinal bars	8 bars, but may be reduced 6 or 4 if clear spacing is less than 150 mm and $N^* \leq 0.1\phi f_c' A_g$	6 bars in a circular arrangement 4 bars in a rectangular arrangement	6 bars in a circular arrangement 4 bars in a rectangular arrangement	Same as nominally ductile (1995)	Same as nominally ductile (1995)
Maximum spacing between longitudinal bars requiring restraint	Circular columns, larger of one quarter of a diameter or 200 mm Rectangular, larger of one third of column dimension in direction of spacing or 200 mm, spacing can be increased in centre of column when $h/b > 20$	Larger of one third of column dimension in direction of spacing or 200 mm for Rectangular column	200 mm	-	-
	For DPRs and LDPRs: Larger of one-quarter of the column dimension (or diameter) in direction of spacing or 200 mm In protected plastic hinge regions and outside plastic hinge regions same as nominally ductile	For DPRs: Larger of one-quarter of the column dimension (or diameter) in direction of spacing or 200 mm			

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
Maximum longitudinal column bar diameter	For DPRs and LDPRs: $\frac{d_b}{h_b} \leq 3.2\frac{\sqrt{f_c'}}{f_y}$ 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 $16\,d_b$ If using $\phi = 0.7$ smaller of $h_{min}, 16d_b$ or $48\,d_s$	For Spirally columns, $d_c/6$ For tied columns, min $\{h_{min}, 16d_b$ and $48\,d_s\}$	For Spirally columns, max {1 in. and $3d_s$}, min {3 in and $d_c/6$ and also $\rho_s > 0.004A_g$}
		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 $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}}\frac{s}{d_b}$ Spirals or hoops of circular shape $\rho_s = \frac{A_{st}}{155d''}\frac{f_y}{f_{yt}}\frac{1}{db}$	Rectangular hoops and ties $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}}\frac{s}{d_b}$ Spirals or hoops of circular shape $\rho_s = \frac{A_{st}}{155d''}\frac{f_y}{f_{yt}}\frac{1}{db}$	-	-	-

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
Confinement reinforcement (outside of the potential plastic hinge region)	Rectangular hoops and ties A_{sh}[3] Spirals or hoops of circular shape ρ_{s}[4]	Rectangular hoops and ties A_{sh}[9] Spirals or hoops of circular shape ρ_{s}[10]	If using φ=0.9 then for Rectangular hoops and ties A_{sh}[11] Spirals or hoops of circular shape ρ_{s}[12]	Spirals shape ρ_{s}[12]	-
Minimum shear reinforcement (outside of the potential plastic hinge region)	$A_{\mathrm{v}} = \frac{1}{16}\sqrt{f'_{\mathrm{c}}}\frac{b_{\mathrm{w}}S}{f_{\mathrm{yt}}}$ For DPRs and LDPRs: $A_{\mathrm{v}} = \frac{1}{12}\sqrt{f'_{\mathrm{c}}}\frac{b_{\mathrm{w}}S}{f_{\mathrm{yt}}}$	-	-	-	-
Maximum shear force (outside of the potential plastic hinge region)	V_{n} $\leq 0.2 f'_{\mathrm{c}} b_{\mathrm{w}} d, or\ 8\ b_{\mathrm{v}}$	-	-	-	-
Minimum diameter for transverse reinforcement (within potential plastic hinge region)	Same as outside plastic hinge region	-	-	Same as outside plastic hinge region	-
Maximum vertical spacing of ties (within potential plastic hinge region)	For DPRs: Smallest of $h_{\mathrm{min}}/4$ or 6 d_{b} For LDPRs: Smallest of $h_{\mathrm{min}}/4$ or 10 d_{b}	For DPRs: Smallest of $h_{\mathrm{min}}/4$ or 6 d_{b}	For DPRs: Smaller of $h/5$, diameter, /5 $6d_{\mathrm{b}}$ or $200\ mm$	Same as outside plastic hinge region	-
Anti-buckling reinforcement (within potential plastic hinge region)	For DPRs and LDPRs: For rectangular hoops and ties $A_{\mathrm{te}} = \frac{\sum A_{\mathrm{b}} f_{\mathrm{y}}}{96 f_{\mathrm{yt}}}\frac{S_{\mathrm{h}}}{d_{\mathrm{b}}}$ For spirals or hoops of circular shape $\rho_{\mathrm{s}} = \frac{A_{\mathrm{st}}}{110d''}\frac{f_{\mathrm{y}}}{f_{\mathrm{yt}}}\frac{1}{db}$	For DPRs: For rectangular hoops and ties $A_{\mathrm{te}} = \frac{\sum A_{\mathrm{b}} f_{\mathrm{y}}}{96 f_{\mathrm{yt}}}\frac{S_{\mathrm{h}}}{d_{\mathrm{b}}}$ For spirals or hoops of circular shape $\rho_{\mathrm{s}} = \frac{A_{\mathrm{st}}}{110d''}\frac{f_{\mathrm{y}}}{f_{\mathrm{yt}}}\frac{1}{db}$	-	-	-
Confinement reinforcement (within potential plastic hinge region)	For DPRs and LDPRs: Rectangular hoops and ties for DPRs, A_{sh}[7]	For DPRs: Rectangular hoops and ties for DPRs, A_{sh}[7]	For DPRs: Spirals or hoops of circular shape ρ_{s1}[14] or ρ_{s2}[15]	Same as outside plastic hinge region	-

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
	for LDPRs, 0.7 A_{sh}[7] Spirals or hoops of circular shape for DPRs, ρ_s[8] for LDPRs, 0.7 ρ_s[8]	for LDPRs, 0.7 A_{sh}[7] Spirals or hoops of circular shape for DPRs, ρ_s[8] for LDPRs, 0.7 ρ_s[8]	Rectangular hoops and ties A_{sh1}[16] or A_{sh2}[17]		
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. $N_{n,max} = \alpha_1 f'_c(A_g - A_{st}) + f_y A_{st}$
2. $N_0^* = 0.7\phi N_{n,max}$
3. $A_{sh} = \frac{(1-\rho_t m)S_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0065 S_h h''$ (N^* = design axial load at ultimate limit state)
4. $\rho_s = \frac{(1-\rho_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0084$ (N^* = design axial load at ultimate limit state)
5. DPR = Ductile Potential Plastic Region
6. $LDPR$ = Limited Ductile Potential Plastic Region
7. $A_{sh} = \frac{(1.3-\rho_t m)S_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N_0^*}{\phi f'_c A_g} - 0.006 S_h h''$
8. $\rho_s = \frac{(1.3-\rho_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N_0^*}{\phi f'_c A_g} - 0.0084$
9. $A_{sh} = \frac{(1-\rho_t m)S_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0065 S_h h''$ $\left(N^* = 0.85\phi\alpha_1 f'_c(A_g - A_{st}) + f_y A_{st}\right)$
10. $\rho_s = \frac{(1-\rho_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0084$ $\left(N^* = 0.85\phi\alpha_1 f'_c(A_g - A_{st}) + f_y A_{st}\right)$
11. $A_{sh} = 0.3 S_h h''\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$
12. $\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$
13. P_e = Maximum design axial load in compression at a given eccentricity
14. $\rho_{s1} = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 0.125\frac{P_e}{\phi f'_c A_g}\right)$
15. $\rho_{s2} = 0.12\frac{f'_c}{f_{yh}}\left(0.5 + 0.125\frac{P_e}{\phi f'_c A_g}\right)$
16. $A_{sh} = 0.3 S_h h''\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$
17. $A_{sh} = 0.12 S_h h''\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$
18. $P_0 = \phi\left(0.85 f'_c(A_g - A_{st}) + A_{st} f_y\right)$

C5E.3 Beam-Column Joints

The following table summarises the evolution of standards-based design/details requirements for beam-column joints, from NZS 3101:1982 to NZS 3101:2006.

Table C5E.3: Evolution of standards-based beam-column joints design/details requirements (Cuevas et al., 2015)

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982
Maximum horizontal shear stress	$v_{jh} \leq \begin{cases} 0.20f'_c \\ 10\text{MPa} \end{cases}$	$v_{jh} \leq 0.25f'_c$	
Minimum horizontal transverse confinement reinforcement	For spirals or circular hoops: $\rho_s = \frac{(1-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0084$ $\rho_s = \frac{A_{st}}{155d''}\frac{f_y}{f_{yt}}\frac{1}{d_b}$ For rectangular hoop and tie reinforcement: $A_{sh} = \frac{(1-p_t m)s_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0065 s_h h''$ $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}}\frac{s_h}{d_b}$ With $\begin{cases} A_g/A_c \leq 1.50 \\ p_t m \leq 0.40 \end{cases}$ (reduce by half when joints connecting beams at all four column faces)	For spirals or circular hoops: $\rho_s = \frac{(1-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0084$ $\rho_s = \frac{A_{st}}{155d''}\frac{f_y}{f_{yt}}\frac{1}{d_b}$ For rectangular hoop and tie reinforcement: $A_{sh} = \frac{(1-p_t m)s_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0065 s_h h''$ $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}}\frac{s_h}{d_b}$ With $\begin{cases} A_g/A_c \leq 1.20 \\ p_t m \leq 0.40 \end{cases}$ (reduce by half when joints connecting beams at all four column faces)	For spirals or circular hoops: $\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$ For rectangular hoop and tie reinforcement: $A_{sh} = 0.3 s_h h''\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$ where $f_{yh} \leq 500MPa$
Spacing limits	$S_{max} = \begin{cases} (D,b,h)/3 \\ 10d_b \\ 200\ mm \end{cases}$	$S_{max} = \begin{cases} (D,b,h)/3 \\ 10d_b \\ 200\ mm \end{cases}$	$S_{max} = \begin{cases} (D,b,h)/5 \\ 10d_b \\ 200\ mm \end{cases}$
Design yield strength (for earthquake)	$f_{yh} \leq 500\text{MPa}$ $f_{yv} \leq 500\text{MPa}$	$f_{yh} \leq 500\text{MPa}$ $f_{yv} \leq 500\text{MPa}$	
Maximum horizontal shear stress (for earthquake)	$v_{jh} \leq \begin{cases} 0.20f'_c \\ 10\text{MPa} \end{cases}$	$v_{jh} \leq 0.20f'_c$	$v_{jh} \leq 1.5\sqrt{f'_c}$
Minimum horizontal joint reinforcement (for earthquake)	For spirals or circular hoops: $\rho_s = \frac{(1.3-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{f'_c A_g} - 0.0084$ (*) $\rho_s = \frac{A_{st}}{110d''}\frac{f_y}{f_{yt}}\frac{1}{d_b}$	For spirals or circular hoops: $\rho_s = 0.70\left\{\frac{(1.3-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{f'_c A_g} - 0.0084\right\}$ $\rho_s = \frac{A_{st}}{110d''}\frac{f_y}{f_{yt}}\frac{1}{d_b}$	For spirals and circular hoops, the greater of: $\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982
	For rectangular hoop and tie reinforcement: $A_{sh} = \frac{(1.3-p_t m)s_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{f'_c A_g} - 0.006 s_h h''$ (*) $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}}\frac{s_h}{d_b}$ With $\begin{cases} A_g/A_c \le 1.50 \\ p_t m \le 0.40 \end{cases}$ (*) 70% reduction for limited ductile	For rectangular hoop and tie reinforcement: $A_{sh} = 0.70\left\{\frac{(1.3-p_t m)s_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{f'_c A_g} - 0.006 s_h h''\right\}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}}\frac{s_h}{d_b}$ With $\begin{cases} A_g/A_c \le 1.20 \\ p_t m \le 0.40 \end{cases}$ (*) 70% reduction not allowed at the joint of the columns of the first storey	$\rho_s = 0.12\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$ For rectangular hoop and tie reinforcement, the greater of: $A_{sh} = 0.3 s_h h''\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$ $A_{sh} = 0.12 s_h h''\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$
Spacing limits (for earthquake)	$S_{max} = \begin{cases} (D,b,h)/4 \\ 6d_b \\ 200\ mm \end{cases}$ (ductile) $S_{max} = \begin{cases} (D,b,h)/4 \\ 10d_b \\ 200\ mm \end{cases}$ (limited ductile)	$S_{max} = \begin{cases} (D,b,h)/4 \\ 6d_b \\ 200\ mm \end{cases}$	$S_{max} = \begin{cases} (D,b,h)/5 \\ 6d_b \\ 200\ mm \end{cases}$
Spacing limits for vertical reinforcement (for ductile members adjacent to the joint)	$S_{max} = \begin{cases} (D,h,b)/4 \\ 200\ mm \end{cases}$ (at least one intermediate bar in each side of the column in that plane)	$S_{max} = \begin{cases} (D,h,b)/4 \\ 200\ mm \end{cases}$ (at least one intermediate bar in each side of the column in that plane)	$S_{max} = 200\ mm$ (at least one intermediate bar in each side of the column in that plane)
Maximum diameter of longitudinal beam bars passing through joints (for ductile members adjacent to the joint)	$\frac{d_b}{h_c} \le 3.3\alpha_f\alpha_d\frac{\sqrt{f'_c}}{1.25 f_y}$ $f'_c \le 70MPa$ $\alpha_d = 1.00$ (ductile) $\alpha_d = 1.20$ (limited ductile)		
Maximum diameter of column bars passing through joints (for ductile members adjacent to the joint)	$\frac{d_b}{h_b} \le 3.2\frac{\sqrt{f'_c}}{f_y}$ (1) $\frac{d_b}{h_b} \le 4.0\frac{\sqrt{f'_c}}{f_y}$ (2) For columns designed by Method B or by Method A (and the joint is below the mid height of the second storey)		

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982
	For columns designed by Method A and the joint is above the mid height of the second storey		

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.

C5E.4 Walls

Table C5E.4 summarises the evolution of the New Zealand standards-based design/details requirements for walls, while Table C5E.5 provides a key to the notation used throughout the various Standards.

Table C5E.4: Evolution of standards-based design/details requirements for walls

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P: 1970	NZS 1900:1964 (bylaw)
Minimum thickness-general	100 mm	100 mm for the uppermost 4 m of wall height and for each successive 7.5 m downward (or fraction thereof), shall be increased by 25 mm	150 mm for the uppermost 4 m of wall height and for each successive 7.5 m downward (or fraction thereof), shall be increased by 25 mm	6 in.	5 in.
Limitations on the height to thickness ratio	If $N^* > 0.2f'_c A_g$ $\frac{K_e L_n}{t} \leq 30$ L_n: the clear vertical distance between floors or other effective horizontal lines of lateral support	If $N^* > 0.2f'_c A_g$ $\frac{L_n}{t} \leq 25$	$\frac{L_n}{t} \leq 10$ 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	$\frac{L_n}{t} \leq 35$ L_n: the distance between lateral supports (Horizontal or Vertical)	$\frac{L_n}{t} \leq 24$ L_n: the distance between lateral supports (Horizontal or Vertical)
Singly reinforced walls Limitations on the height to thickness ratio to prevent flexural torsional buckling of in-plane loaded walls	$\frac{k_{ft}L_n}{t} \leq 12\sqrt{\frac{L_n/L_w}{\lambda}}$ where: $N^* \leq 0.015f'_c A_g$ and $\frac{L_n}{t} \leq 75$ and $\frac{k_{ft}L_n}{t} \leq 65$	No limitations	No limitations	No limitations	No limitations
Doubly reinforced walls Moment magnification required when:	$\frac{k_e L_n}{t} \geq \frac{\alpha_m}{\sqrt{\frac{N^*}{f'_c A_g}}}$	No requirements	No requirements	No requirements	No requirements

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P: 1970	NZS 1900:1964 (bylaw)
Minimum thickness for prevention of instability within plastic hinge region	$b_{\mathrm{m}} = \frac{\alpha_{\mathrm{r}} k_{\mathrm{m}} \beta (A_{\mathrm{r}} + 2)\mathrm{L_w}}{1700\sqrt{\xi}}$ $\beta = 7\ (DPR)$ $\beta = 5\ (LDPR)$	$b_{\mathrm{m}} = \frac{k_{\mathrm{m}} (\mu + 2)(A_{\mathrm{r}} + 2}{1700\sqrt{\xi}}$	No requirements	No requirements	No requirements
Ductile detailing length - special shear stress limitations	$\max\{L_{\mathrm{w}}, 0.17\frac{M}{V}\}$ Measured from the 1st flexural yielding section Need not be greater than $2L_{\mathrm{w}}$	$\max\{L_{\mathrm{w}}, \frac{h_{\mathrm{w}}}{6}\}$ Measured from the 1st flexural yielding section Need not be greater than $2L_{\mathrm{w}}$	$\max\{L_{\mathrm{w}}, \frac{h_{\mathrm{w}}}{6}\}$ Measured from the 1st flexural yielding section Need not be greater than $2L_{\mathrm{w}}$	No requirements	No requirements
Limitation on the use of singly reinforced walls	$\rho_{\mathrm{l}} \leq 0.01$ $b \leq 200$ mm	$b \leq 200$ mm $\mu \leq 4$	$b \leq 200$ mm or if the design shear stress $\leq 0.3\sqrt{f_c'}$	Earth retaining walls: $b < 10$ in. Other walls: $b < 9$ in.	$t < 10$ in.
Minimum longitud-inal reinforce-ment ratio	$\rho_{\mathrm{n}} = \frac{\sqrt{f_c'}}{4f_{\mathrm{y}}}$	$\rho_{\mathrm{l}} = \frac{0.7}{f_{\mathrm{y}}}$	$\rho_{\mathrm{l}} = \frac{0.7}{f_{\mathrm{y}}}$	$\frac{9000}{f_y}\% \geq 0.18\%$ Note: f_y in units of [psi]	0.0025 (mild steel) 0.0018 (high tensile steel)
Maximum longitud-inal reinforce-ment ratio (ρ_l)	$\frac{16}{f_y}$	$\frac{16}{f_y}$	$\frac{16}{f_y}$	No requirements	No requirements
Maximum spacing of longitudinal reinforcement	Min $\{Lw/3, 3t$, or 450 mm$\}$	Min $\{2.5b, 450\ \mathrm{mm}\}$	Min$\{2.5b, 450\ \mathrm{mm}\}$	Min $\{2.5b, 18\mathrm{in.}\ (457\ \mathrm{mm})\}$	$2.5b$
Anti-buckling reinforce-ment (Outside of the potential plastic hinge region)	Where: $\rho_{\mathrm{l}} > \begin{Bmatrix} \frac{2}{f_y}\ \mathrm{DPR} \\ \frac{3}{f_y}\ \mathrm{LDPR} \end{Bmatrix}$ $d_{\mathrm{tie}} > d_{\mathrm{b}}/4$ Spacing $< 12d_{\mathrm{b}}$	Where: $\rho_{\mathrm{l}} > \frac{2}{f_y}$ $d_{\mathrm{tie}} > d_{\mathrm{b}}/4$ Spacing $< 12d_{\mathrm{b}}$	Hoop or tie sets Spacing $\leq \min \begin{cases} \text{least lateral dime} \\ 16d_{\mathrm{b}} \\ 48d_{\text{transverse b}} \end{cases}$	No requirements	No requirements

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P: 1970	NZS 1900:1964 (bylaw)
Anti-buckling reinforce-ment (Within the potential plastic hinge region)	Where: $\rho_l > \begin{Bmatrix} \frac{2}{f_y}\ DPR \\ \frac{3}{f_y}\ LDPR \end{Bmatrix}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b}$ Spacing ≤ $\begin{Bmatrix} 6d_b\ (DPR) \\ 10d_b\ (LDPR) \end{Bmatrix}$	Where: $\rho_l > \frac{2}{f_y}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b}$ Spacing ≤ $6d_b$	Where: $\rho_l > \frac{2}{f_y}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{100}$ Spacing ≤ $6d_b$	No requirements	No requirements
Confine-ment reinforce-ment	Where neutral axis depth $> c_c = \frac{0.1\phi_{ow}L_w}{\lambda}$ $\lambda = 1.0$ (DPR) $\lambda = 2.0$ (LDPR) $A_{sh} = \alpha s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} \left(\frac{c}{L_w} - 0.07\right)$ $\alpha = 0.25$ (DPR) $\alpha = 0.175$ (LDPR)	Where neutral axis depth $> c_c = \left(\frac{0.3\phi_o}{\mu}\right) L_w$ $A_{sh} = \left(\frac{\mu}{40} + 0.1\right) s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} (- 0.07)$	Where neutral axis depth $> c_c = \begin{Bmatrix} 0.1\phi_o S l_w \\ \text{or} \\ \frac{8.6\phi_o S l_w}{(4-0.7S)\left(17+\frac{h_w}{l_w}\right)} \end{Bmatrix}$ $A_{sh} = \max \begin{cases} 0.3 s_h h'' \left(\frac{A_g^*}{A_c^*} - 1\right) \frac{f}{f} \\ 0.12 s_h h'' \frac{f_c'}{f_{yh}} (\end{cases}$	No requirements	No requirements
Maximum spacing of confine-ment reinforce-ment	DPR: $\min\{6d_b, 0.5t\}$ LDPR: $\min\{10d_b, t\}$	$\text{Min}\ \{6d_b, 0.5t, 150\ \text{mm}\}$	$\text{Min}\ \{6d_b, 0.5t, 150\ \text{mm}\}$	No requirements	No requirements
Minimum confine-ment length	Max $\begin{Bmatrix} c - 0.7c_c \\ 0.5c \end{Bmatrix}$ c: neutral axis depth	Max $\begin{Bmatrix} c - 0.7c_c \\ 0.5c \end{Bmatrix}$ c: neutral axis depth	$0.5c$	No requirements	No requirements
Maximum nominal shear stress	$v_n \le 0.2f'_c$ or 8 MPa	$v_n \le \begin{cases} 0.2f'_c \\ 1.1\sqrt{f'_c} \\ 9\ \text{MPa} \end{cases}$	$v_n \le 0.2f'_c$ or 6 MPa	$v_u \le (0.8 + 4.6\frac{H}{D})\phi\sqrt{f'_c}$ $v_u \le 5.4\phi\sqrt{f'_c}$ for $H/D < 1$ $v_u \le 10\phi\sqrt{f'_c}$ for $H/D > 2$ $\phi = 0.85$	$v = \frac{f_c}{1 + \frac{h^2}{49t^2}}$
Concrete shear strength (simplified)	$V_c = \min \begin{cases} 0.17\sqrt{f} \\ 0.17\left[\sqrt{f'_c}\right. \end{cases}$	$v_c = \min \begin{cases} 0.2\sqrt{f'_c} \\ 0.2\left[\sqrt{f'_c} + \right. \end{cases}$	$v_c = \min \begin{Bmatrix} 0.2\sqrt{f'_c} \\ 0.2\left[\sqrt{f'_c} + \frac{P_u}{A_g}\right] \end{Bmatrix}$	The shear stress carried by the concrete shall not exceed:	No requirements

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P: 1970	NZS 1900:1964 (bylaw)
				$v_c = \left(3.7 - \frac{H}{D}\right) 2\phi\sqrt{f'_c}$ $v_c \leq 5.4\phi\sqrt{f'_c}$ for $H/D < 1$ $v_c \leq 2\phi\sqrt{f'_c}$ for $H/D > 2.7$ $\phi = 0.85$	
Shear reinforce-ment	$A_v = V_s \frac{s_2}{f_{yt} d}$	$A_v = \frac{(v_n - v_c) b_w s_2}{f_{yt}}$	$A_v = \frac{(v_n - v_c) b_w s_2}{f_{yh}}$	$A_v = \frac{V'_u s}{\phi f_y d \left(\frac{H}{D} - 1\right.}$	No requirements
Minimum shear reinforce-ment	$A_v = \frac{0.7\, b_w\, s_2}{f_{yt}}$	$A_v = \frac{0.7\, b_w\, s_2}{f_{yt}}$	$A_v = \frac{0.7\, b_w\, s_2}{f_{yh}}$	$A_v = \frac{V'_u s}{\phi f_y d}$ or Ratio (%): $\frac{9000}{f_y} \geq 0.18$	0.0025 (mild steel) 0.0018 (high tensile steel)
Maximum spacing of shear reinforce-ment	$\mathrm{Min}\left(\frac{L_w}{5}, 3t, \text{or } 450\right.$	$\mathrm{Min}\left(\frac{L_w}{5}, 3t, \text{or } 450\right.$	$\mathrm{Min}\left(\frac{L_w}{5}, 3t, \text{or } 450\text{ mm}\right)$	$2.5t$, 18in (457 mm)	$2.5t$
Vertical reinforce-ment	$\rho_n \geq \frac{0.7}{f_{yn}}$ Spacing $\leq \min\{\frac{L_w}{3}, 3t,$ 450 mm$\}$	$\rho_n \geq \frac{0.7}{f_{yn}}$ Spacing $\leq \min\{\frac{L_w}{3}, 3t,$ 450 mm$\}$	$\rho_n \geq \frac{0.7}{f_{yn}}$ Spacing $\leq \min\{\frac{L_w}{3}, 3t,$ 450 mm$\}$	No requirements	No requirements
Maximum shear strength provided by the concrete in ductile detailing length	$V_c = \left(0.27\lambda\sqrt{f'_c} + \frac{N^*}{4A_g}\right) b_w d \geq 0.0$ $\lambda = 0.25DPR$ $\lambda = 0.5LDPR$	V_c shall not be taken larger than: $v_c = 0.6\sqrt{\frac{N^*}{A_g}}$ Total nominal shear stress shall not exceed: $v_n = \left(\frac{\phi_{ow}}{\mu} + 0.15\right)\sqrt{f'_c}$	V_c shall not be taken larger than: $v_c = 0.6\sqrt{\frac{P_e}{A_g}}$ Total nominal shear stress shall not exceed: $v_n = (0.3\phi_o S + 0.16)\sqrt{f'_c}$ S: structural type factor as defined by NZS 4203	No requirements	No requirements

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P: 1970	NZS 1900:1964 (bylaw)
Splicing of flexural tension reinforcement	One-third (*DPR*) and one-half (*LDPR*) of reinforcement can be spliced where yielding can occur	One-third of reinforcement can be spliced where yielding can occur	One-third of reinforcement can be spliced where yielding can occur	One-half of reinforcement can be spliced where yielding can occur	No requirements
Maximum compressive stress in concrete	No requirements	No requirements	No requirements	$\left[1-\left(\frac{h}{35d}\right)^3\right]0.2f'_c$ h: distance between supports d: thickness of wall	Direct loading: kf_{cu} $k=\frac{p}{5}-0.007\frac{h}{t}+0.2$ f_{cu}: minimum crushing strength P: total percentage of vertical reinforcement $0.25 \le p \le 0.5$ $\frac{h}{t} \ge 10$ 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 C5E.5: Notation used in New Zealand standards for walls

Notation	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970*	NZS 1900:1964 (bylaw)
Design axial load at the ultimate limit state	N^*	N^*	P_u	N/A	N/A
The clear vertical distance between floors or other effective horizontal lines of lateral support, or clear span	L_n	L_n	L_n	h	h
Wall thickness	t, b	b	b	d, b	t
Effective length factor for Euler buckling	k_e	N/A	N/A	N/A	N/A
Effective length factor for flexural torsional buckling	k_{ft}	N/A	N/A	N/A	N/A
Horizontal length of wall	L_w	L_w	l_w	D	N/A
Thickness of boundary region of wall at potential plastic hinge region	b_m	b_m	N/A	N/A	N/A
Total height of wall from base to top	h_w	h_w	h_w	H	N/A
Aspect ratio of wall (h_w/L_w)	A_r	A_r	N/A	N/A	N/A
Yield strength of non-prestressed reinforcement	f_y	f_y	f_y	f_y	N/A
Yield strength of transverse reinforcement	f_{yh}	f_{yh}	f_{yh}	N/A	N/A
Yield strength of shear reinforcement	f_{yt}	f_{yt}	f_{yh}	f_y	N/A
Ratio of vertical (longitudinal) wall reinforcement area to gross concrete area of horizontal section	$\rho_n = \frac{A_t}{A_g}$	N/A	N/A	N/A	N/A
The ratio of vertical wall reinforcement area to unit area of horizontal gross concrete section	$\rho_l = \frac{A_s}{ts_v}$	$\rho_l = \frac{A_s}{bs_v}$	$\rho_l = \frac{A_s}{bs_v}$	N/A	N/A
Diameter of longitudinal bar	d_b	d_b	d_b	N/A	N/A
Centre-to-centre spacing of shear reinforcement along member	s	s	s	N/A	N/A
Computed distance of neutral axis from the compression edge of the wall section	c	c	c	N/A	N/A
A limiting depth for calculation of the special transverse reinforcement	c_c	c_c	c_c	N/A	N/A
Overstrength factor	ϕ_{ow}	ϕ_o	ϕ_o	N/A	N/A
Area of concrete core	${A_c}^*$	${A_c}^*$	${A_c}^*$	N/A	N/A
Gross area of concrete section	${A_g}^*$	${A_g}^*$	${A_g}^*$	N/A	N/A

Notation	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970*	NZS 1900:1964 (bylaw)
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	μ	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 C5F Diaphragms Grillage Modelling/Analysis Methodology

C5F.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).

C5F.2 Grillage Section Properties

Grillage members are typically modelled as concrete elements, without reinforcement modelled, in an elastic analysis program. Figure C5F.1 illustrates a grillage model developed for a complicated podium diaphragm.

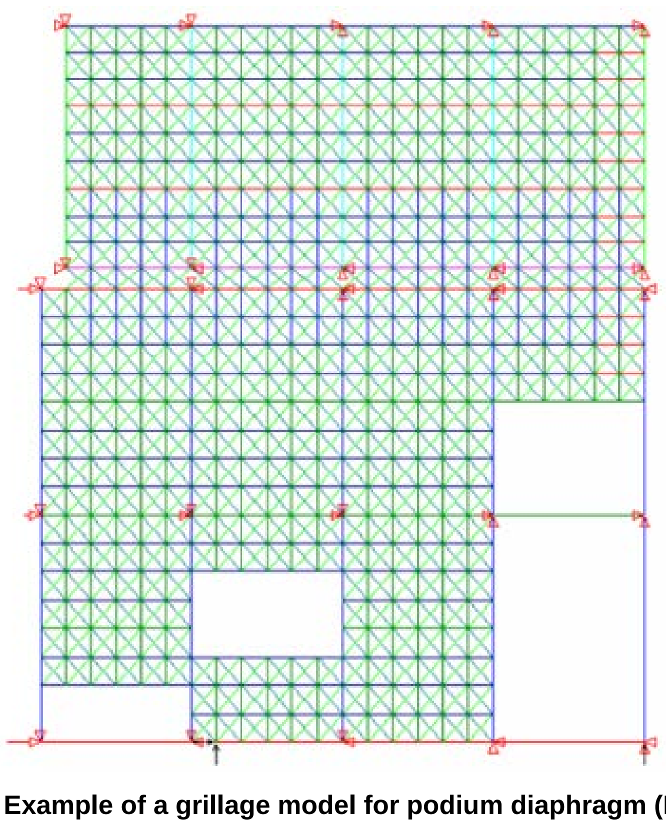


Figure C5F.1: Example of a grillage model for podium diaphragm (Holmes, 2015)

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 C5F.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).

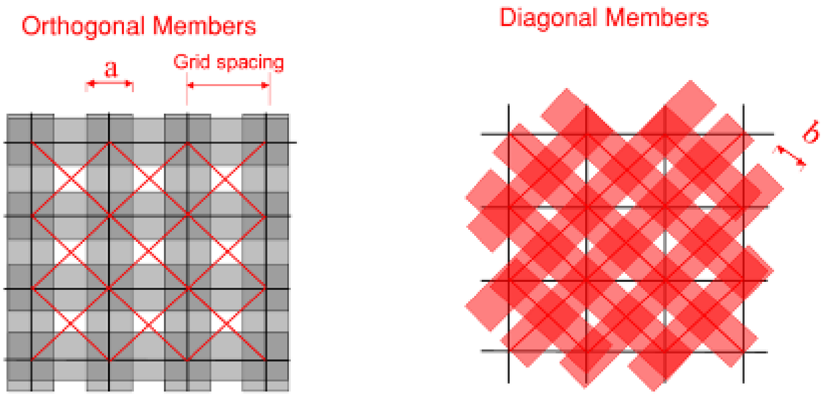


Figure C5F.2: Grillage beam dimensions for the square grillage (Hrennikoff, 1941)

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.

C5F.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 x grid spacing
  - carries both tension and compression forces
- Diagonal members:
  - width B = 0.53 x grid spacing
  - carries compression forces only.

C5F.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.

C5F.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

- 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.

C5F.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).

C5F.7 Loss of Load Paths due to Diaphragm Damage

Modify the grillage to account for anticipated diaphragm damage/deterioration. For example, where floor to beam separation similar to that illustrated in Figure C5F.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 C5F.3).

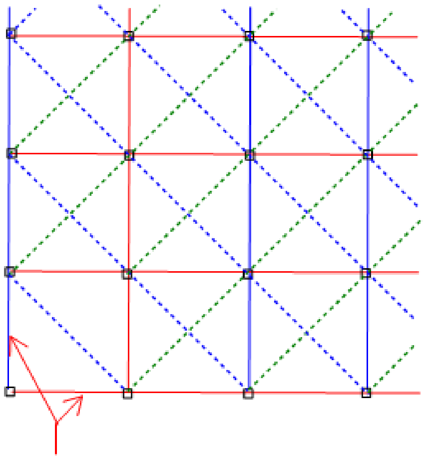


Figure C5F.3: Recommended grillage modelling at corner columns when frame elongation is anticipated (Holmes, 2015)

C5F.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.

C5F.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 C5F.4, are equal to the difference in shears in vertical lateral load resisting elements above and below the diaphragm being assessed.

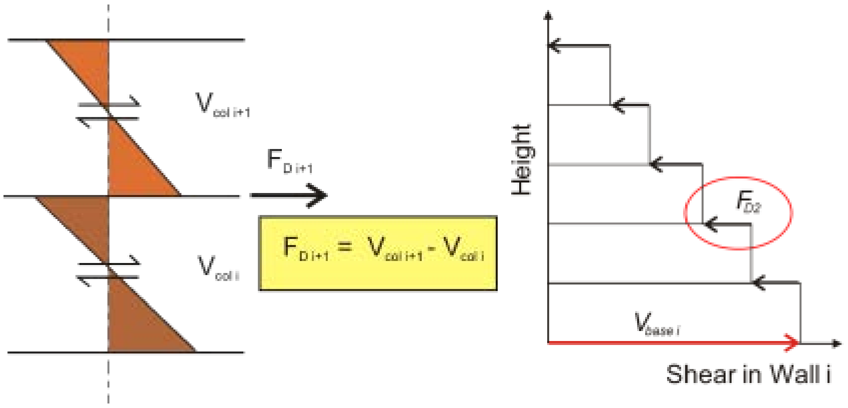


Figure C5F.4: Floor forces, F_{Di}, determined from pESA (Holmes, 2015)

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 C5F.5). Care is required to ensure that sign conventions (i.e. input and output of actions) are maintained.

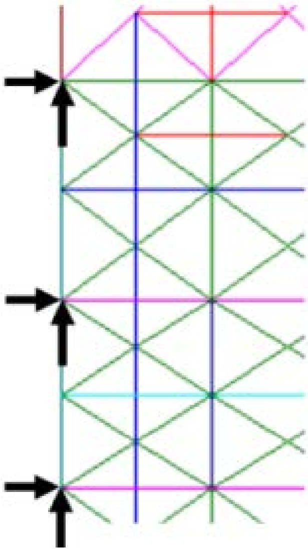


Figure C5F.5: Floor forces F_{Di} in both X and Y directions at nodes connected to vertical elements – for one direction of earthquake attack (Holmes, 2015)

C5F.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, $F_{\mathrm{OP,i}}$, 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, $F_{\mathrm{OP,i}}$, 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, $F_{\mathrm{OP,i}}$, determined accordingly (i.e. taking the difference in out-of-plane shear in the vertical elements above and below the diaphragm being assessed).

C5F.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 C5G Deformation Capacity of Precast Concrete Floor Systems

C5G.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 C5G.1 and C5G.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).

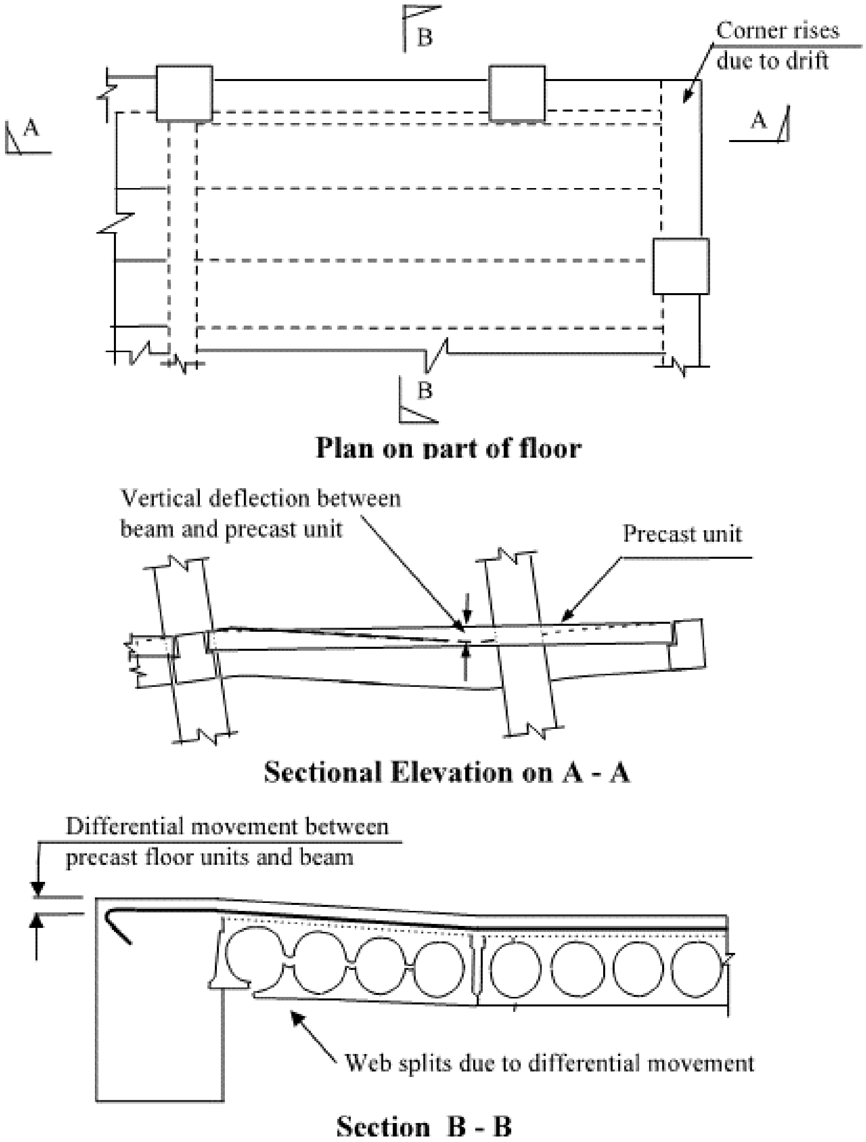


Figure C5G.1: Incompatible displacements between precast floor units and beams (Fenwick et al., 2010)

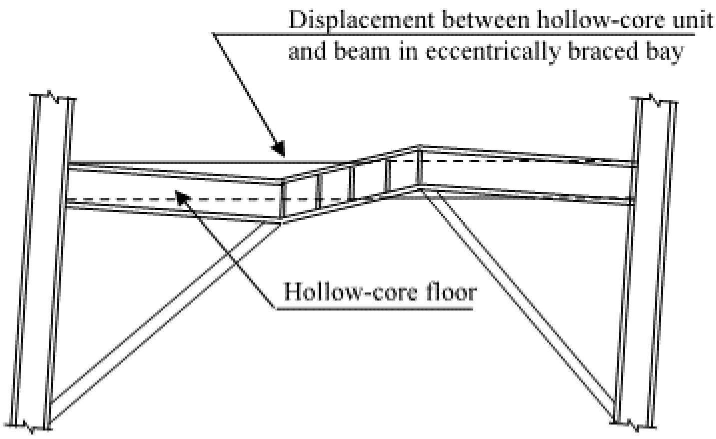


Figure C5G.2: Incompatible displacements between precast floor units and braced bay (Fenwick et al., 2010)

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 C5G.3 and C5G.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 C5G.3(c).

C5G.2 Extent of Diaphragm Cracking

Figures C5G.3 and C5G.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 C5G.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.

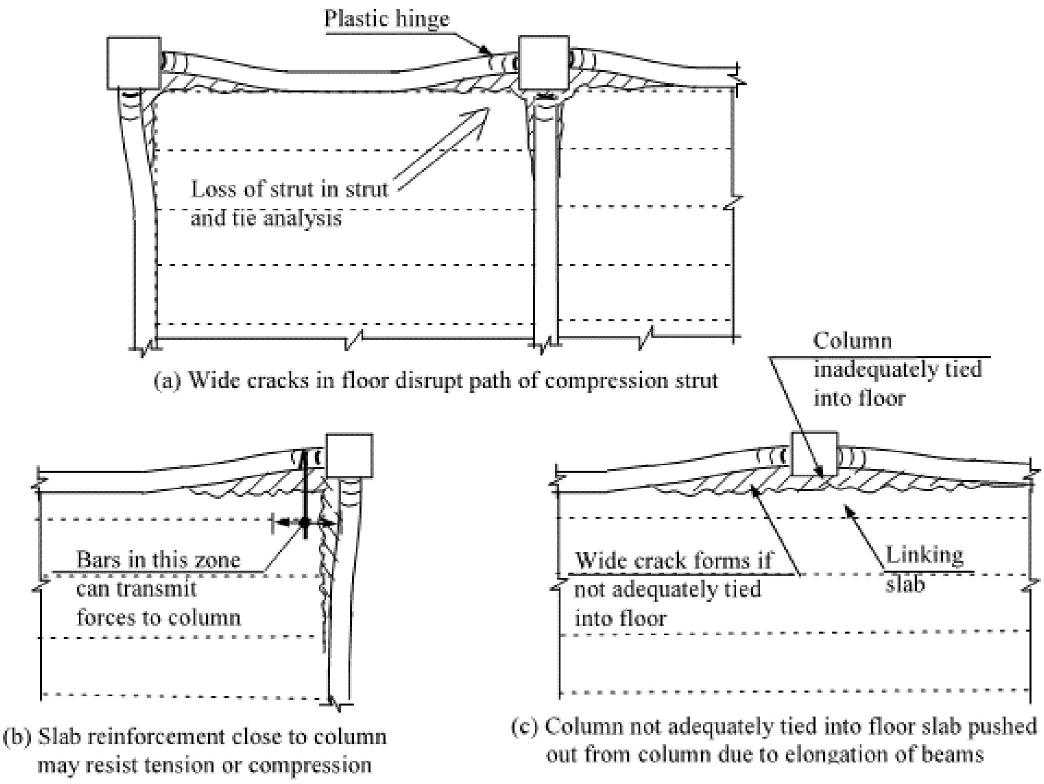
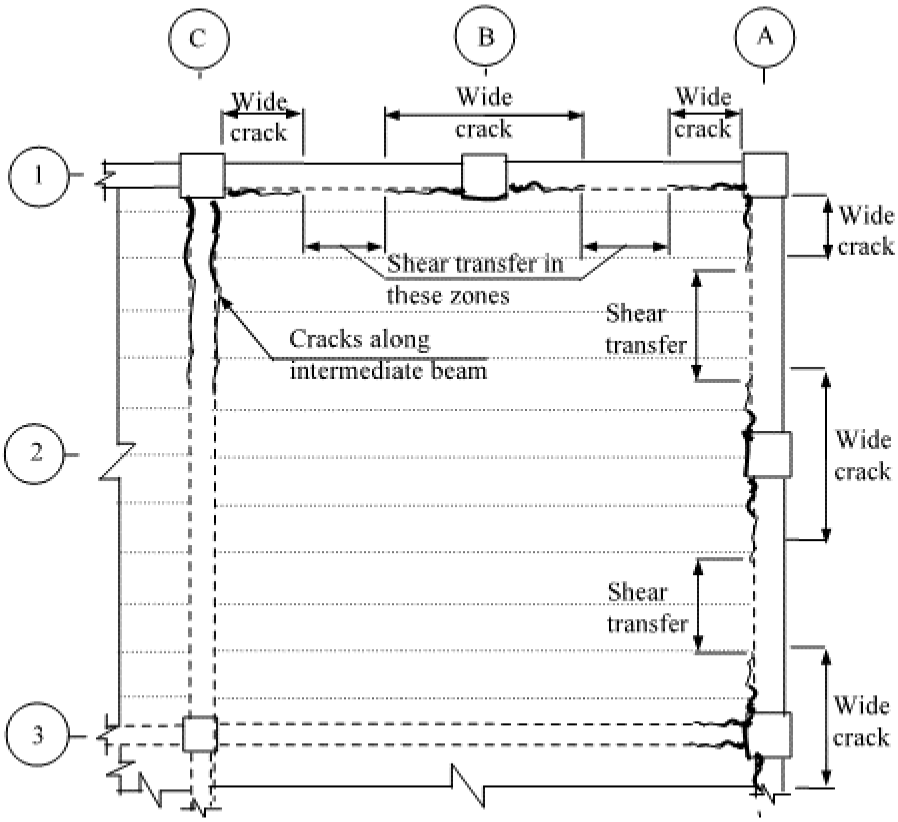


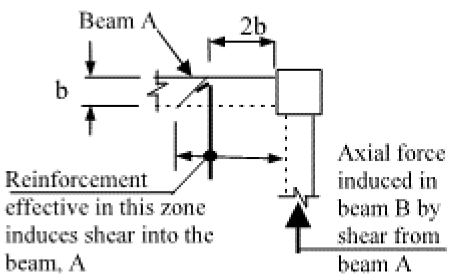
Figure C5G.3: Separation crack between floor and supporting beam due to frame elongation (Fenwick et al., 2010)

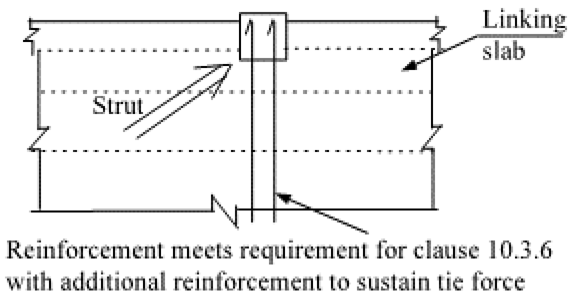
Figure C5G.4: Observed separation between floor and supporting beam due to frame elongation in 2011 Canterbury earthquakes (Des Bull)

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.



(a) Plan on part of a floor showing areas where shear can be transferred to perimeter frames





(b) Effective zone for reinforcement acting near a column

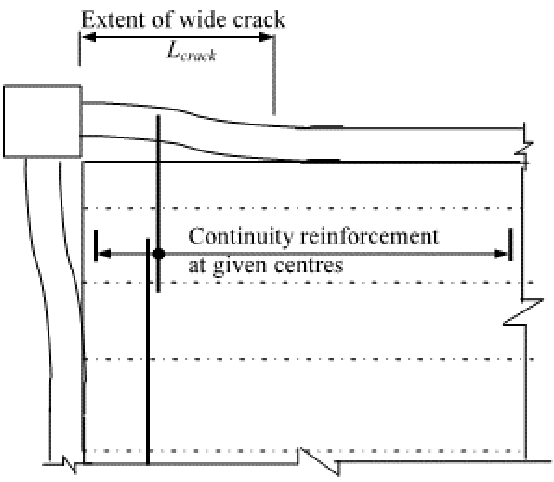
(c) Intermediate column acts as node for strut and tie forces to transfer shear to frame

Figure C5G.5: Location of cracks and strut and tie forces in a diaphragm (Fenwick et al., 2010)

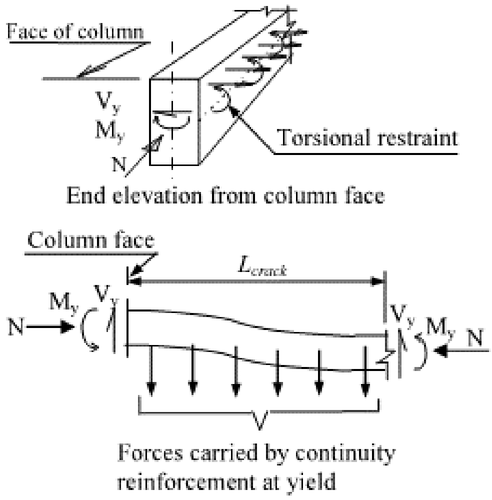
The extent of cracking along an intermediate beam, such as the beam on line C in Figure C5G.5 depends on the relative magnitudes of inelastic deformation sustained in the perimeter frame (such as the frame on line 1) and an adjacent intermediate frame (such as frame on line 3 in Figure C5G.5(a)). Where 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.

C5G.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 C5G.6 shows the separation of a corner column due to elongation in beams framing into the column.



(a) Plan on floor showing separation of beams from floor



(b) Plan of length of wide crack

Figure C5G.6: Separation between floor and supporting beam (Fenwick et al., 2010)

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_{crack}, is given by:

$$L_{\mathrm{crack}} = \sqrt{\frac{2M_{\mathrm{o}}}{F}} \qquad \ldots\text{C5G.1}$$

where:

M_{o}	=	flexural overstrength of beam about the vertical axis
F	=	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 C5G.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.

C5G.4 Inter-storey Drift Capacity of Diaphragm Components

C5G.4.1 General

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

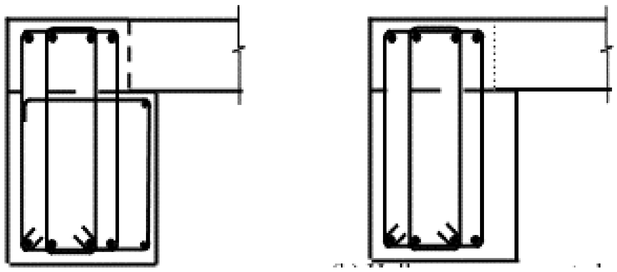
- loss of support of precast floor units, and
- failure of precast floor units due to seismic actions, including the consideration of incompatible displacements.

C5G.4.2 Loss of support

Overview

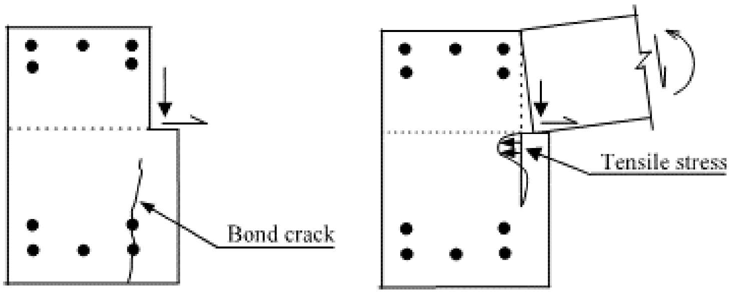
There are two key aspects to consider when assessing precast concrete floor units for loss of support:

- 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 C5G.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 C5G.8.



(a) Support ledge tied into beam **(b) Hollowcore supported on cover concrete**

Figure C5G.7: Support on concrete ledge tied into the supporting element or on cover concrete (Fenwick et al., 2010)



(a) Bond cracks **(b) Prying action**

Figure C5G.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 hollowcore 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 C5G.9.

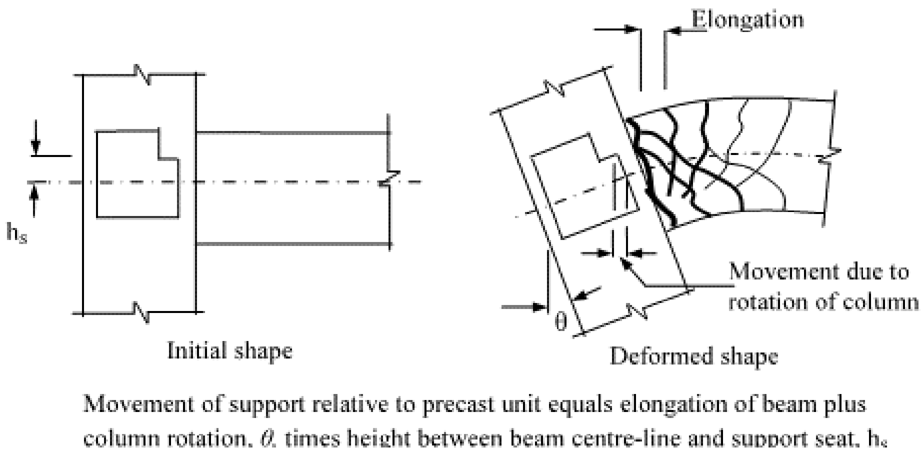


Figure C5G.9: Displacement at support of precast unit due to elongation and rotation of support beam (Fenwick et al., 2010)

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 C5G.10 illustrates three plastic hinge elongation types.

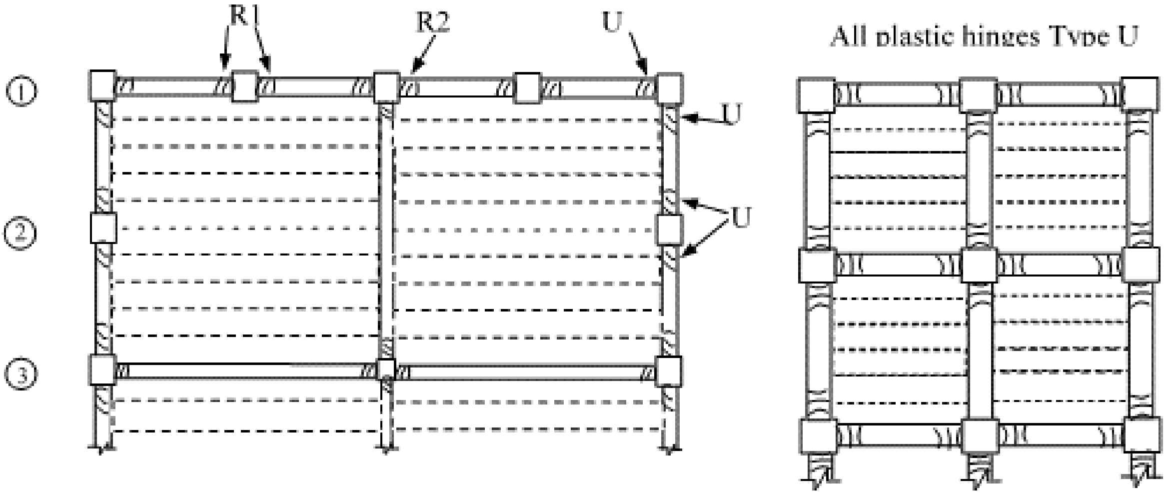


Figure C5G.10: Part plan of floor showing plastic hinge elongation types U, R1 and R2 (Fenwick et al., 2010)

For type U and R1 plastic hinges little restraint is provided by the floor slab and the elongation at mid-depth of the beam, Δ_L, can be calculated as:

$$\Delta_L = 0.0014 h_b \frac{\phi_u}{\phi_y} \leq 0.037 h_b \quad \text{...C5G.2}$$

where:

h_b	=	beam depth
ϕ_y	=	beam first yield curvature
ϕ_u	=	ultimate curvature demand on beam determined using plastic hinge lengths specified in Section C5.5.2.2.

For type R2 plastic hinges where there is a transverse beam framing into the column the elongation at mid-depth of the beam, Δ_L, can be calculated in accordance with Equation C5G.3 where the terms are as defined above:

$$\Delta_L = 0.0007 h_b \frac{\phi_u}{\phi_y} \leq 0.02 h_b \quad \text{...C5G.3}$$

Equations C5G.4 and C5G.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:

$$\Delta_{\mathrm{g}} = \left(\frac{h_{\mathrm{b}}}{2} - h_{\mathrm{L}}\right)\theta \qquad \text{…C5G.4}$$

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)

θ = beam rotation.

Total movement of precast floor unit units relative to the ledge providing support due to elongation and rotation of support beams, Δ_{rot}, is calculated as:

$$\Delta_{\mathrm{rot}} = \Delta_{\mathrm{L}} + \Delta_{\mathrm{g}} \qquad \text{…C5G.5}$$

where:

Δ_{L} and Δ_{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 hollowcore 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, Δ_{spall}, is given by:

$$\Delta_{\mathrm{spall}} = 0.5L_{\mathrm{s}} \leq 35\ mm \qquad \text{C5G.6}$$

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 C5G.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 hollowcore 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 hollowcore 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 hollowcore 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.

C5G.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 C5H Buckling of Vertical Reinforcement and Out-of-Plane Instability in Shear Walls

This appendix outlines a possible approach to assessing buckling of reinforcing bars in RC elements with emphasis on shear walls. It also provides background information on the out-of-plane instability of shear walls.

C5H.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 el al., 1999; Bae et al., 2005; Urmson 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 ($\varepsilon_{\mathrm{s,cr}}$) on the maximum tensile strain experienced by the bar before the cycle reversal takes place ($\varepsilon_{\mathrm{st}}$) 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 C5H.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 ($\varepsilon_{\mathrm{s}} = \varepsilon_{\mathrm{st}}$, $f_{\mathrm{s}} = f_{\mathrm{st}}$). Two strain levels are used for the same point, representing large and moderate initial elongations of the steel: $\varepsilon_{\mathrm{st}}$ = 4.0% and $\varepsilon_{\mathrm{st}}$ = 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 ($\varepsilon_{\mathrm{s}} = \varepsilon_0^+$, $f_{\mathrm{s}} = 0$).

The strain associated with point 2, ε_0^+, can be estimated using Equation C5H.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 C5H.1 becomes Equation C5H.2

$$\varepsilon_0^+ = \varepsilon_{\mathrm{st}} - f_{\mathrm{st}}/E_{\mathrm{s}} \quad \text{...C5H.1}$$

$$\varepsilon_0^+ = \varepsilon_{\mathrm{st}} - \varepsilon_{\mathrm{y}} \quad \text{...C5H.2}$$

From point 2 towards point 3, the zero strain point (ε_{s} = 0, fs < 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, ε_p^*, can be calculated with

Equation C5H.3 (Rodriguez et al., 2013), as a function of the restraining ratio s_v/d_b. If the critical buckling strain, $\varepsilon_{s,cr}$, is defined with reference to the zero strain axis, it can be calculated with Equation C5H.4.

$$\varepsilon_p^* = \frac{11-(5/4)(s_v/d_b)}{100} \qquad \text{...C5H.3}$$

$$\varepsilon_{s,cr} = \varepsilon_{st} - \varepsilon_y - \varepsilon_p^* \qquad \text{...C5H.4}$$

Consider, as an example, the damage developed at the free end of the walls W1 and W2 presented in Figure C5H.1. These walls were part of two buildings constructed in Christchurch and were damaged during the 22 February 2011 earthquake.

In Figure C5H.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.

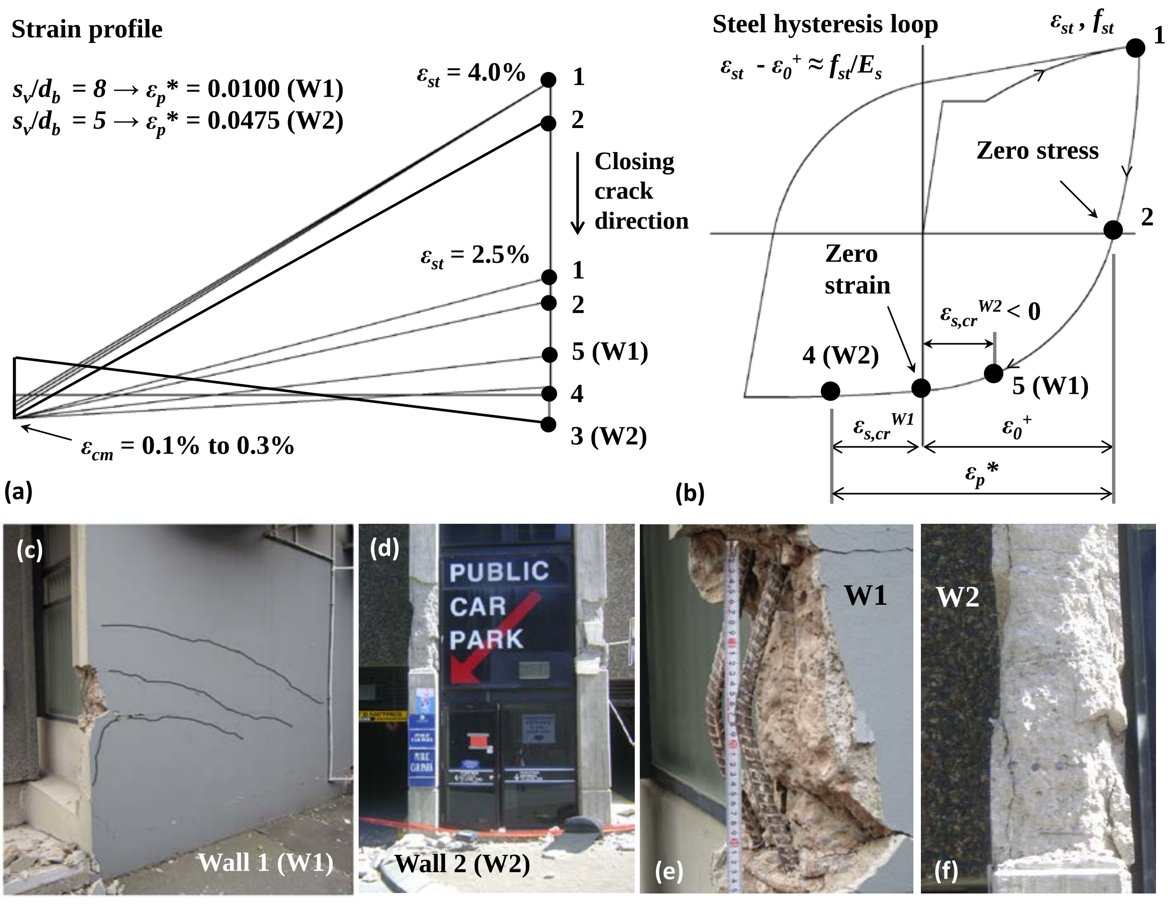


Figure C5H.1: Buckling critical strain definition (Quintana-Galo et al. 2016)

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 with the ultimate curvature, $\varepsilon_{\mathrm{sm}}$ to obtain $\varepsilon_{\mathrm{s,cr}}$, such that $\varepsilon_{\mathrm{st}} = \varepsilon_{\mathrm{sm}}$ in Equation C5H.5. The strain $\varepsilon_{\mathrm{s,cr}}$ should be compared with the maximum compression strain in the inverse direction of the moment. If the section is symmetric in geometry and reinforcement, $\varepsilon_{\mathrm{s,cr}}$, can be compared with the maximum compression strain in the concrete at $\varepsilon_{\mathrm{cu}}$ or $\varepsilon_{\mathrm{cu,c}}$ as corresponds, which is the strain that governs in most of the cases. If the maximum strain of the steel $\varepsilon_{\mathrm{sm}}$ 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 $\varepsilon_{\mathrm{sm}}$ smaller, but closer to 6%, $\varepsilon_{\mathrm{s,cr}}$ takes positive values, indicating that buckling will occur while the bar experiences tensile strains (point 5 in Figure C5H.1).
- The second approach is to set the ultimate strain of the concrete as $\varepsilon_{\mathrm{cm}} = \varepsilon_{\mathrm{cu}}$ or $\varepsilon_{\mathrm{cu,c}}$ as corresponds, and calculate the maximum tensile strain $\varepsilon_{\mathrm{st}} = \varepsilon_{\mathrm{su,b}}$, 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 $\varepsilon_{\mathrm{s,cr}} = -\varepsilon_{\mathrm{cm}}$, and $\varepsilon_{\mathrm{st}} = \varepsilon_{\mathrm{su,b}}$ in Equation C5H.5:

  $$\varepsilon_{\mathrm{su,b}} = -\varepsilon_{\mathrm{cm}}^{\mathrm{r}} + \varepsilon_{\mathrm{y}} + \varepsilon_{\mathrm{p}}^{*} \qquad \ldots\text{C5H.5}$$

  The superscript r is used to indicate that this concrete strain corresponds to the cross section under reversed actions.

Note:

As a general rule, if the spacing of confinement stirrups is greater than $7d_{\mathrm{b}}$, 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 $s_{\mathrm{v}}/d_{\mathrm{b}}$ (6.5, 10 15) are presented in Figure C5H.2 (Mau and El-Mabsout, 1989). Figure C5H.3 shows the maximum compression normalised stress and the lateral displacement of the bar for different restraining ratios.

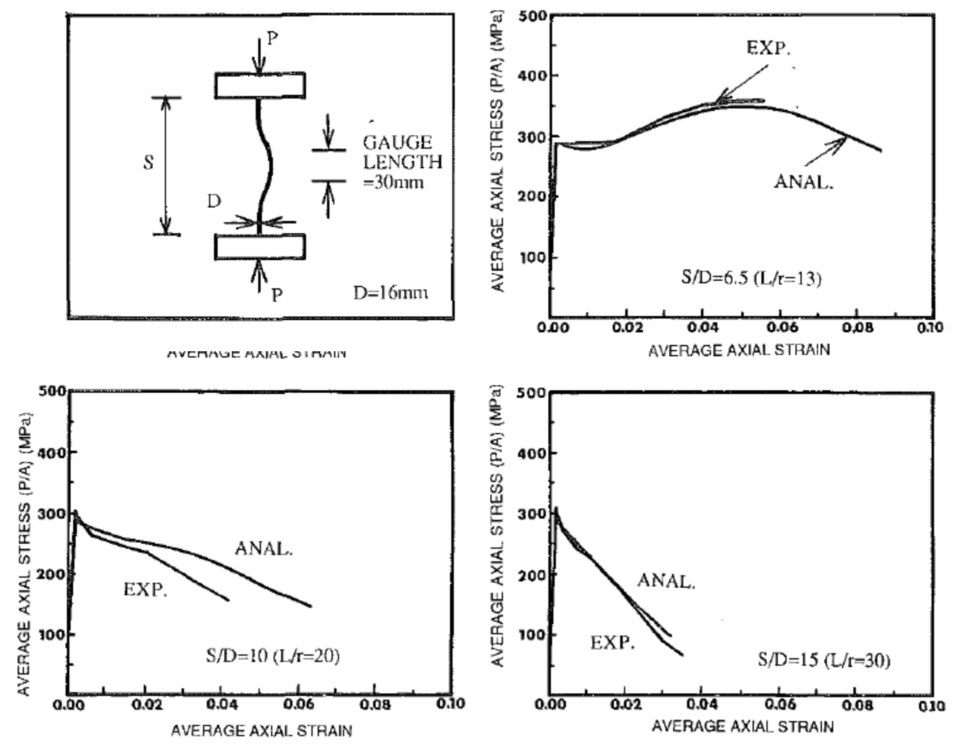


Figure C5H.2: Stress-strain curves of a steel bar in compression for s_v/d_b = 6.5, 10 and 15 (Mau and El-Mabsout, 1989)

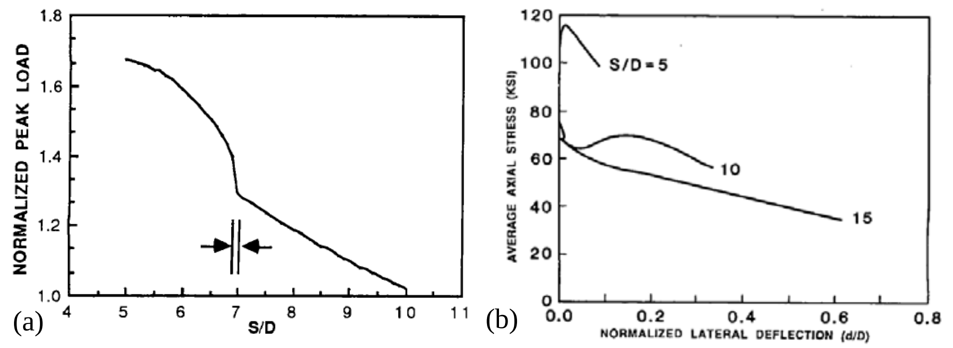


Figure C5H.3: (a) normalised peak load (relative to buckling stress) and critical restraining ratio s_v/d_b = 7, (b) lateral displacement of the bar for different restraining ratios s_v/d_b (Mau, 1990)

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, ε_y= 0.25%, and assuming a well confined concrete ε_{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 ε_{cu} = 1% this would suggest that ε_{su}= 3% represents a simplistic upper limit to be adopted to account for buckling effects in section analysis. This is instead of $\varepsilon_{s,max}$= 6% assumed in Table C5.8 considering a flexural-dominated and ideal behaviour.

Based on Equation C5.9, the **probable curvature at the onset of buckling, $\boldsymbol{\phi^*_{\mathrm{prob}}}$,** and the corresponding plastic displacement, δ^*_p, can be estimated using Equations C5H.6 and C5H.7 respectively.

$$\phi^*_{\mathrm{prob}} = \frac{\varepsilon^*_{\mathrm{p}}}{\gamma l_{\mathrm{w}}} \qquad \text{...C5H.6}$$

$$\delta^*_{\mathrm{prob}} = L_{\mathrm{p}}\left(\phi^*_{\mathrm{prob}} - \phi_{\mathrm{y}}\right)\left(h_{\mathrm{w}} - 0.5L_{\mathrm{p}}\right) \qquad \text{...C5H.7}$$

where:

γl_{w} is shown in Figure C5H.4.

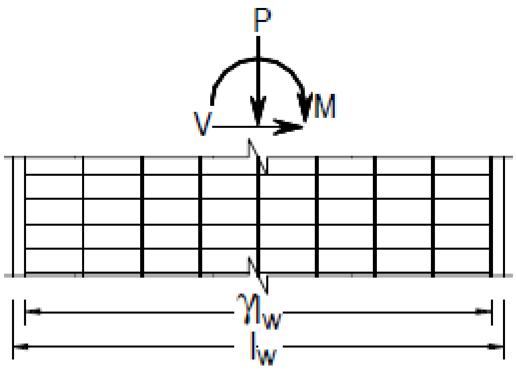


Figure C5H.4: Definition of γl_{w} according to Rodriguez et al. (2013)

C5H.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.

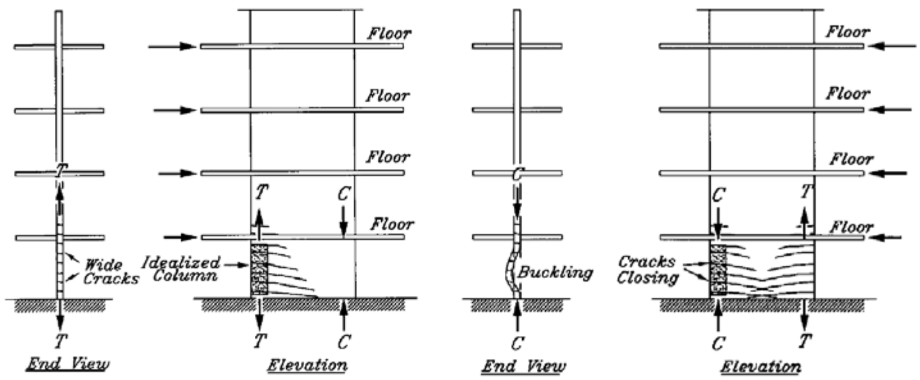
For more detailed information and preliminary results refer to Dashti et al. (2015, 2016).

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 C5H.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.



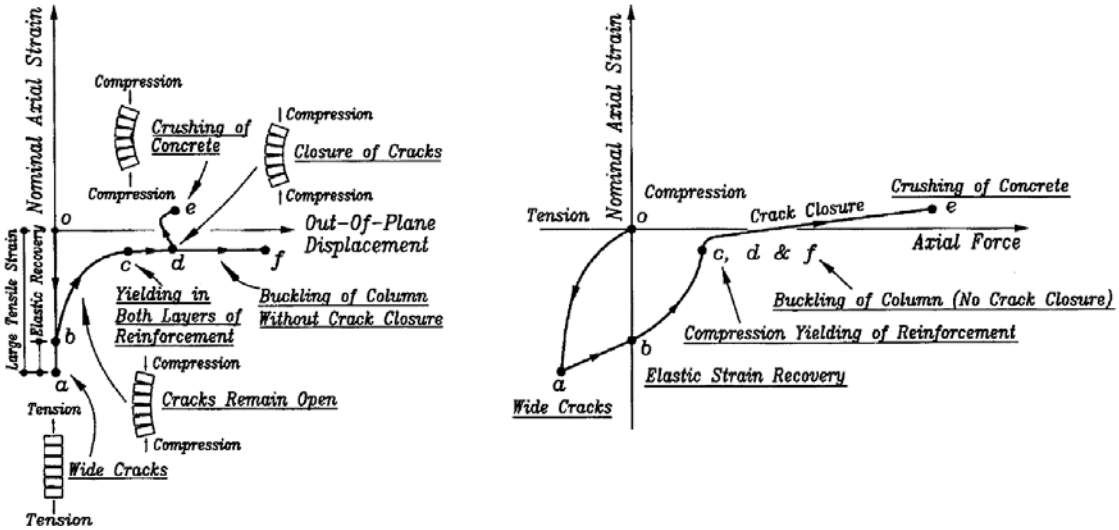
(a) Opening of cracks under tension cycle

(b) Closing of cracks under compression cycle

Figure C5H.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 C5H.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 C5H.1.



(a) nominal axial strain versus out-of-plane displacement

(b) nominal axial strain versus axial force

Figure C5H.6: Axial reversed cyclic response of reinforced concrete slender wall (Chai and Elayer, 1999)

Table C5H.1: Behaviour of wall end-region under the loading cycle shown in Figure C5H.6

	Loading	Unloading	Reloading			
Path	**o-a**	**a-b**	**b-c**	**c-d**	**d-e**	**d-f**
	Large tensile strain	Elastic strain recovery mainly in reinforcing steel	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	Compression yielding in the second layer of the reinforcement, and a rapid increase in the out-of-plane displacement	Closure of cracks at point d and decrease of out-of-plane displacement and increase of out-of-plane displacement after significant compressive strain is developed in the compressed concrete	An excessive crack opening where subsequent compression would not result in the closure of the cracks but a continued increase in the out-of-plane displacement and eventual buckling of the column

As can be seen in Figure C5H.6 and Table C5H.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.

Appendix C5I Procedure for Evaluating the Equivalent "Moment" Capacity of a Joint, M_j

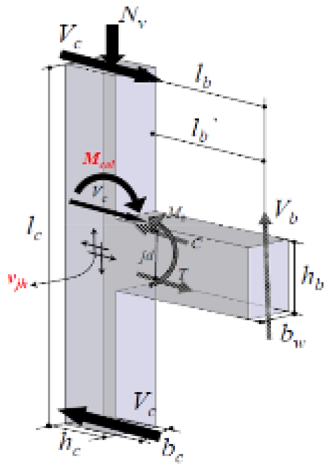
In order to compare the hierarchy of strength and determine the expected sequence of events within beam–column joint subassemblies (refer to Section C5.6.1 for the full procedure) the joint shear capacity can be expressed as a function of a comparable parameter to the capacity of beams and columns.

As a benchmark parameter, it is suggested to take an equivalent moment in the column (based on equilibrium considerations).

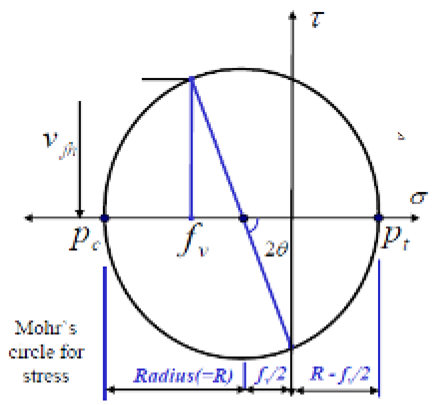
In Table C5I.1 and Figure C5I.1 below the probable shear force $V_{\mathrm{prob,jh}}$ is expressed as a function of the moment in the column, leading to the expression of M_{col} as the equivalent moment in the column corresponding to the given joint parameter.

Table C5I.1: Step-by-step procedure to express the joint capacity as a function of equivalent column moment M_j or M_{col}

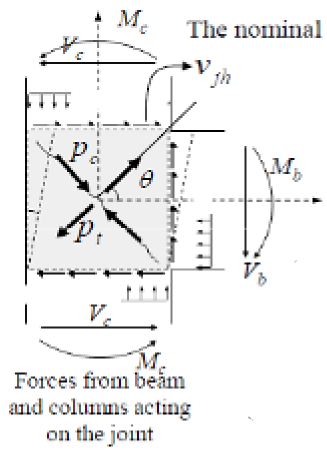
Horizontal shear force acting on the joint core	$V_{jh} = T - V_c$ …C5I.1
Equilibrium of the external action	$V_c l_c = V_b l_b$ …C5I.2
Rearrange to get V_b	$V_b = \frac{V_c l_c}{l_b}$ …C5I.3
Moment acting at the face of the joint core	$M_b = V_b\left(l_b - \frac{h_c}{2}\right) = Tjd$ …C5I.4
Rearrange to get T	$T = \frac{M_b}{jd} = \frac{V_b\left(l_b - \frac{h_c}{2}\right)}{jd} = \frac{V_c l_c\left(l_b - \frac{h_c}{2}\right)}{l_b jd}$ …C5I.5
Substitute into the 1st equation	$V_{jh} = T - V_c = \frac{V_c l_c\left(l_b - \frac{h_c}{2}\right)}{l_b jd} - V_c = V_c\left[\frac{l_c}{l_b jd}\left(l_b - \frac{h_c}{2}\right) - 1\right]$ …C5I.6
Rearrange to get V_c	$V_c = \frac{V_{jh}}{\left[\frac{l_c}{l_b jd}\left(l_b - \frac{h_c}{2}\right) - 1\right]}$ …C5I.7
Joint capacity in terms of the column moment	$M_{col} = V_c\left(\frac{l_c - h_b}{2}\right) = \frac{V_{jh}}{\left[\frac{l_c}{l_b jd}\left(l_b - \frac{h_c}{2}\right) - 1\right]}\left(\frac{l_c - h_b}{2}\right)$ …C5I.8
Assume $j = 0.9d$ and $A_e = b_j \times h_c$	$M_{col} = \frac{\nu_{jh}(1000)}{\phi}\ kNm\ and\ \phi = \frac{2l_b' l_c - 1.8 d l_b}{0.9 d l_b A_e (l_c - h_b)}$ …C5I.9
Nominal horizontal shear stress at the mid-depth of the joint core	$\nu_{jh} = \frac{V_{jh}}{b_j \times h_c}$ …C5I.10
Effective width of the joint	$b_j = \min(b_c, b_w + 0.5h_c)\ if\ b_c \geq b_w$ …C5I.11 $b_j = \min(b_w, b_c + 0.5h_c)\ if\ b_c \leq b_w$ …C5I.12
Principal tensile and compressive stresses	$p_t = p_c = -\frac{f_v}{2} \pm R$ …C5I.13
Substitute $R = \sqrt{\left(\frac{f_v}{2}\right)^2 + \nu_{jh}{}^2}$ from Mohr's Circle Theory	$p_t = -\frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + \nu_{jh}{}^2}$ …C5I.14
Rearrange to get horizontal shear	$\nu_{jh} = \sqrt{p_t{}^2 + p_t f_v}$ …C5I.15
Substitute into the joint capacity equation	$M_{col} = \frac{\sqrt{p_t{}^2 + p_t f_v}(1000)}{\phi}\ kNm$ …C5I.16
Principal tensile stress	$p_t = k\sqrt{f_c'}$ …C5I.17
Stress due to axial load	$f_v = \frac{N_v}{A_e}$ …C5I.18



(a)



(b)



(c)

Figure C5I.1: (a) Free-body diagram of a beam-column joint sub-assembly; (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 C5I.1:

$$V_{\mathrm{jh}} = T + C's - V_{\mathrm{c}} \quad \text{...C5I.19}$$

  assuming $M_{\mathrm{b}} = M_{\mathrm{c}}$ for interior beam-column joints, instead of $M_{\mathrm{b}} = 2M_{\mathrm{c}}$ 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 sub-assembly 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.

Appendix C5J Establishing the 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 C5J.1 shows:

- conventional tensile and compressive flexural failures
- shear capacity/failure and shear degradation at various ductility levels (μ = 2 and μ = 4)
- lap-splice failure of the column longitudinal reinforcement.

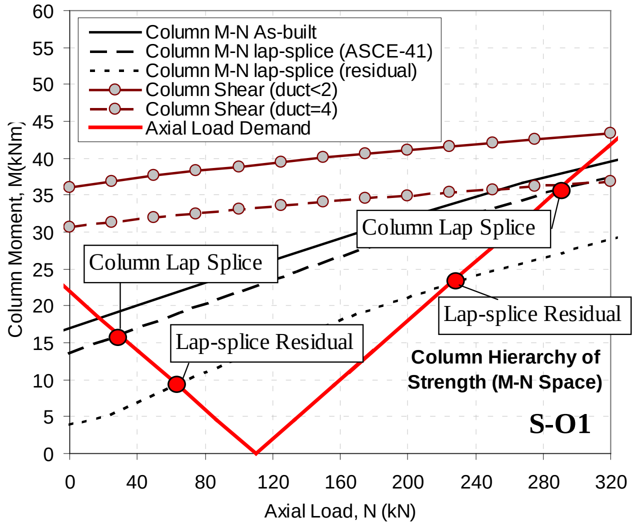


Figure C5J.1: Internal hierarchy of strength of column failure modes within an M-N interaction diagram (Kam, 2011)

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 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 inter-storey drift) level, as shown in Figure C5J.2.

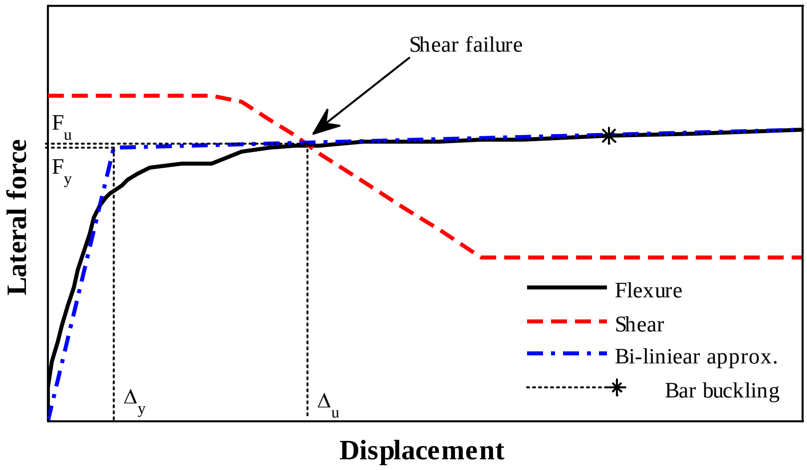


Figure C5J.2: Example of the combined flexural-shear mechanisms within a force-displacement capacity curve for a column (Stirrat et al., 2014)