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RC walls in Australia: seismic design and detailing to AS 1170.4 and AS 3600

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ABSTRACT

This paper reviews the current seismic design procedures for reinforced concrete buildings in Australia, with direct reference to the Australian earthquake loading code, AS 1170.4 and Australian concrete structures standard, AS 3600. The paper aims to clarify seismic design procedures to practising structural engineers and undergraduate students in Australia, whom have had little exposure to seismic design given the lower seismic nature of Australia. The paper begins with a brief history of seismic design in Australia, followed by an overview and discussion of the seismic performance objectives required by the National Construction Code (i.e. Building Code of Australia). Following, a comprehensive discussion of ductility factors for RC wall structures and associated detailed practices to comply with the ductility assumptions assumed when performing analysis procedures set out in AS 1170.4 is presented. The emphasis herein has been placed on issues with the detailing of RC wall structures, as these structures form the majority of low-, mid- and high-rise multi-storey buildings in Australia. Common detailing practices for RC walls that can significantly limit the seismic response are discussed and guidance for designers is presented. The paper is concluded by providing recommendations for the seismic design of RC wall structures in Australia.

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RC walls; seismic design; wall buildings; RC wall detailing

1. Seismic design in Australia

Australia is a region of ‘lower seismicity’ and historically the seismic design and analysis of structures was not required, or where required, was often ignored by designers. This is in stark contrast to regions of ‘higher seismicity’, e.g. New Zealand, Japan or the west coast of the U.S., where seismic design procedures can be traced back to the 1920s or 1930s (Paulay and Priestley 1992). Prior to 1979, no earthquake code or document had been published nor mandated by building authorities in Australia. Australia’s first earthquake loading code, AS 2121 (Standards Association of Australia 1979) was published in 1979, partially in response to the 1968 magnitude 6.8 earthquake in Meckering, Western Australia.

AS 2121 had only limited implications on the design of structures in Australia, with only the South Australian building code and the then Department of Housing and Construction mandating its use, whilst Western Australia required its use for buildings in the Meckering region (Woodside 1992). AS 2121 was not mandated by the remaining building authorities in Australia and hence the majority of buildings constructed in this era would not have been designed for earthquake resistance, much like buildings designed prior to 1979. For

this reason, it was considered that ‘AS 2121 has been on the whole, a non-code, ignored largely by the design community and building regulators’ (Woodside 1992).

The attitude of designers and building professionals in Australia shifted following the 1989 magnitude 5.6 earthquake in Newcastle. This event represented a delimitation in mindset where interest in earthquake engineering design and research in Australia significantly increased. Coincidentally, the code committee for the earthquake loading standard met about two weeks prior to the Newcastle earthquake (Woodside 1992); however, it was the earthquake itself that created real traction and led to the development of a new earthquake loading standard specifically written for Australia, unlike AS 2121, which was largely a reproduction of earthquake standards from the U.S.

The next version of the Australian earthquake loading code was published in 1993 as Part 4 of the AS/NZS 1170 structural design actions series, AS 1170.4 (Standards Australia 1993a). The 1993 version of the code was published in conjunction with a commentary, AS 1170.4 Supp1 (Standards Australia 1993b). The general format of the 1993 version of the code laid the way for the now current version of the code published in 2007 (Standards Australia 2007) and the commentary

which was published shortly thereafter by the Australian Earthquake Engineering Society (Wilson and Lam 2007). The 1993 version of the code was mandated by the Australian Building Code Board (ABCB), but designers could often avoid designing for earthquake actions if their building was deemed 'ductile'. All reinforced and prestressed concrete structures were deemed to be ductile, irrespective of the level of detailing provided, by the 1993 version of the code, which for most buildings at the time, would have been a false assumption. The 2007 version of AS 1170.4 requires all buildings to be designed for earthquake actions and was 'called up' in the Building Code of Australia (National Construction Code 2016) from around 2008.

This lack of historical perspective on seismic design has resulted in an Australian building stock legacy that has not been designed nor assessed for earthquake resistance. It has also contributed to many misconceptions in the design community regarding seismic design, particularly the commonly held belief that if the base shear resulting from wind actions exceeds the base shear from an equivalent static seismic analysis, seismic actions can simply be ignored. The objective of this paper is to better educate designers and improve the seismic design and detailing of RC wall buildings in Australia. The reader should note that non-structural parts and components are also required to be designed for seismic actions and they are directed to Woodside (2016) for a further discussion on the subject.

This paper is the second in a series of publications by the authors on the design of RC walls in Australia and aims to examine and discuss the seismic design procedures of multi-storey buildings in Australia. The emphasis herein is directed towards RC wall buildings as RC rectangular walls and buildings cores are the predominant lateral load resisting system for low, mid and high-rise buildings in Australia (Menegon et al. 2017). The paper aims to firstly provide clarity and background to the current seismic performance objectives in the relevant Australian codes and standards and secondly provide useful discussions and design guidance to aid structural engineers designing and detailing RC wall structures for seismic compliance in Australia.

2. Design methodology and performance objectives

The Building Code of Australia (BCA), published as part of the of the National Construction Code (NCC) series, offers two options to achieve seismic compliance; either a 'deemed-to-satisfy solution' or 'performance solution'. The deemed-to-satisfy solution for an RC building requires a designer to determine the relevant earthquake design actions using the Australian Standard for earthquake actions, AS 1170.4 (Standards Australia 2007) and then assess the structural resistance of the building using the Australian Standard for concrete structures,

AS 3600 (Standards Australia 2009). The performance solution for an RC building allows a designer to work outside the relevant Australian Standards; however, due to the vague and non-descript performance criteria set out by the BCA, additional advice from expert consultants would typically be required and generally could not be justified, or seen as impractical, for normal building projects. Prior to discussing the specific performance objectives for Australia, typical seismic performance objectives for regions of higher seismicity will be discussed for context.

2.1. Performance-based seismic design

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

The building performance levels are usually expressed by easy to understand terms such as: level 1, no damage or operational; level 2, repairable damage; and level 3, near collapse or collapse prevention during a maximum considered earthquake (MCE) event. The level of seismic ground shaking is usually expressed by a return period (RP) event that is calculated based on a probability of exceedance across the design life of the building. Priestley, Calvi, and Kowalsky (2007) and Sullivan, Priestley, and Calvi (2012) recently proposed a matrix of seismic performance criteria for buildings. The latter has been presented in Figure 1.

Sullivan, Priestley, and Calvi (2012) proposes that two zones be established: Zone A, moderate to high seismicity, where Levels 1, 2 and 3 performance criteria shall be met; and Zone B, low seismicity, where only Level 3 performance criteria shall be met. In regions of higher seismicity, it is likely that Level 1 or 2 will govern the design. In regions of lower seismicity where earthquakes are rare and the difference between low and high RP events can be significant (as highlighted later in Figure 4), Sullivan, Priestley, and Calvi (2012) recommend the focus should be placed on solely meeting the Level 3 criteria to ensure loss of life is prevented under long return period events.

There is a general consensus amongst earthquake engineers that a MCE event for normal buildings (i.e. Importance Level 2 buildings in the Australian context) should have a 2500-year RP (rounded up from 2475-year), which corresponds to a probability of exceedance (PE) of 2% in 50 years (i.e. there is a 2% chance a

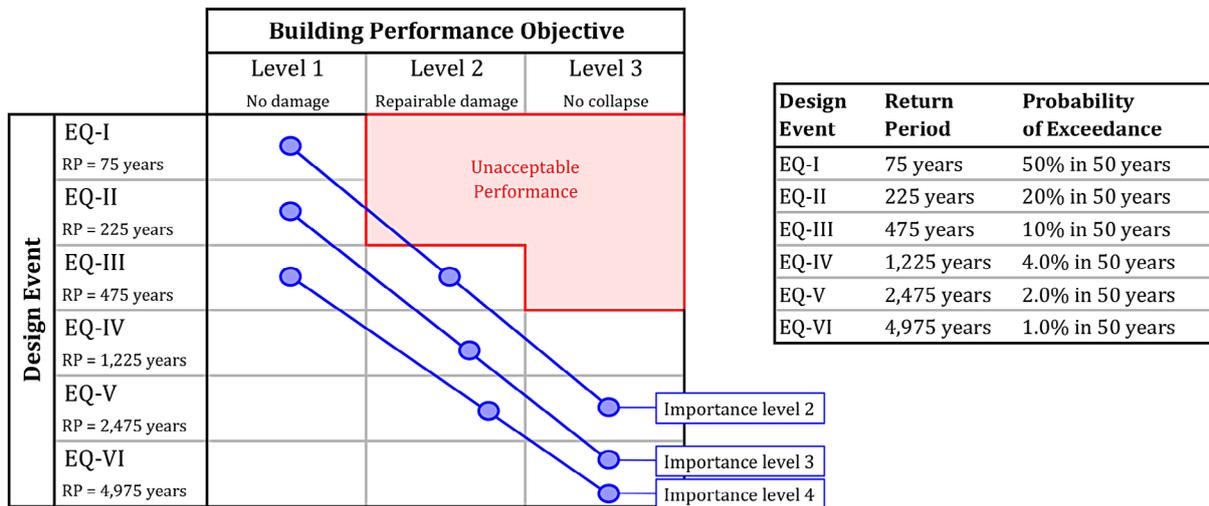


Figure 1. Sullivan, Priestley, and Calvi (2012) proposed seismic performance objectives.

building with a design life of 50 years would undergo MCE level ground accelerations during its life span).

The old version of ASCE/SEI 7-05 (American Society of Civil Engineers 2005), which outlines the minimum design loads for buildings (including seismic provisions) and is referenced by the International Building Code (IBC) for use in California and many other states in the US, specifies that a probabilistic MCE event shall have a PE of 2% in 50 years (i.e. a RP of 2500 years). It is noted that since the 2012 edition of the IBC (International Code Council (ICC) 2012) and the release of ASCE/SEI 7-10 (American Society of Civil Engineers 2010), the hazard definition has changed from a PE of 2% in 50 years to a risk-targeted performance requirement where there is a 1% chance of collapse in 50 years. However, despite this change in hazard definition, the primary intended performance of the former has been retained (Building Seismic Safety Council 2009).

A PE of 2% in 50 years (or 2500-year RP MCE event) has also been adopted in the National Building Code of Canada (National Research Council of Canada 2015) and the U.K. National Annex to Eurocode 8 (British Standards Institute 2008). Furthermore, Tsang (2014) presents a concise argument for adopting a 2500-year RP MCE event for the seismic design of buildings in Hong Kong and other regions of lower seismicity, which would include Australia. Tsang (2014) argues that a maximum allowable annual fatality risk (i.e. reliability level) of a building occupant during an earthquake should be in the order of the 10^{-6} and to achieve such a reliability level, a collapse prevention performance objective must be met under a 2500-year RP MCE event.

2.2. Seismic performance objectives in Australia

The seismic performance objectives for Australia will be discussed alongside the seismic performance objectives for New Zealand. Many Australian standards are published as joint Australian and New Zealand Standards

and as such, the two countries share many similarities in terms of design procedures, methodologies, general terminology and compliance procedures. Additionally, the New Zealand city of Christchurch in 2011 experienced a devastating and extremely damaging earthquake that was characteristic of the typical earthquake faulting mechanism and magnitude that could occur in Australia (Goldsworthy 2012; Hare 2014). The Christchurch earthquake resulted in an estimated 1000 buildings being partially or fully demolished, a total cost in excess of NZ\$40B and a death toll of 185 persons, most of which resulted from the complete structural collapse of the CTV tower and the Pyne Gould Corporation building (Hare 2014). The common terminology and shared design codes (albeit only some) and the hindsight developed regarding the severity of low magnitude 'bullseye' events such as Christchurch, makes New Zealand an ideal example for contrasting Australia, a region of lower seismicity, against a region of higher seismicity.

The BCA does not provide clear seismic performance objectives for Australian buildings. Instead, it provides vague performance metrics that could easily be subject to extremely varying interpretations within the engineering community. The performance requirements from the BCA (Section BP1.1(a)) are summarised below:

- (a) A building or structure, during construction and use, with appropriate degrees of reliability, must
 - (i) perform adequately under all reasonably expected design actions; and
 - (ii) withstand extreme or frequently repeated design actions; and
 - (iii) be designed to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage; and
 - (iv) avoid causing damage to other properties,

The vague performance metrics in the BCA – whether intentional or not – impel designers to adopt the prescriptive deemed-to-satisfy approach, in which designers are instructed to use the joint Australian and New Zealand standard: *Structural design actions, Part 0: General principles*, AS/NZS 1170.0 (Standards Australia and Standards New Zealand 2002). AS/NZS 1170.0 is written as a ‘means for demonstrating compliance with the requirements of Part B1 of the Building Code of Australia’ and establishing ‘compliance with Clause B1 “Structure” of the New Zealand Building Code (NZBC)’.

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

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

The Importance Level definitions and associated return periods for each limit state objective for buildings in New Zealand are outlined in AS/NZS 1170.0 and have been summarised in Figure 2.

AS/NZS 1170.0 requires all buildings in Australia to be designed for ULS earthquake actions based on a given RP for that building’s Importance Level. The Importance Level and RP for earthquake actions are both specified in the BCA. The respective seismic hazard for a given return period can then be determined using AS 1170.4. AS 1170.4 also requires designers to perform a ‘special study’ to ensure that Importance Level 4 buildings remain serviceable (i.e. operational) after the equivalent design event for an Importance Level 2 building. The seismic performance objectives for Australian buildings are summarised in Figure 3.

It should be noted from Figures 2 and 3 that for normal buildings (i.e. Importance Level 2), both the Australian and New Zealand seismic performance

objectives require that a ULS design check be performed for only a 500-year RP event. AS/NZS 1170.0 defines ULS as ‘states associated with collapse, or with similar forms of structural failure’, implying that ULS in the Australian and New Zealand design context relates to a collapse prevention performance objective. This suggests that the Australian seismic design approach is somewhat in line with what is proposed by Sullivan, Priestley, and Calvi (2012) for a region of lower seismicity; however, the RP event that is being designed for is significantly lower than what is proposed there and in other international codes (i.e. a 2500-year RP event).

Similarly, in New Zealand it appears that for normal buildings the MCE event with a 500-year RP is significantly lower than what it should be. However, it was seen following the Christchurch earthquake, which was considered to be in excess of a 2500-year RP event (Goldsworthy 2012; Kam and Pampanin 2011; Tsang 2011), that the majority of the building stock achieved a collapse prevention performance criteria. This is due in part to the conservative nature of using force-based (or strength-based) design methods for ensuring collapse of a building does not occur. This is discussed in Volume 3 of the Canterbury Earthquakes Royal Commission (Canterbury Earthquakes Royal Commission 2012) where it is argued that the conservative nature of ULS design provides the building with adequate reserve strength to achieve adequate performance during a larger event. More specifically, ‘the conservative aspects of designing for ULS (that is using the lower characteristics material strengths, strength reduction factors, etc.) gives a structure protection against collapse in an earthquake above the ULS design level of shaking’. These conservative aspects of force-based ULS design and a deeply engrained design philosophy in New Zealand material codes which strives for a fully ductile structural system by adopting strict capacity design principles led to adequate performance under what would be considered a ‘real’ MCE event (i.e. an event with a PE of 2% in 50 years).

Capacity design is a design philosophy where the designer ‘tells the structure how to behave’ by establishing relative strength hierarchies in the structural elements. Further, it aims to create such a strength hierarchy in the building that while the maximum lateral load capacity

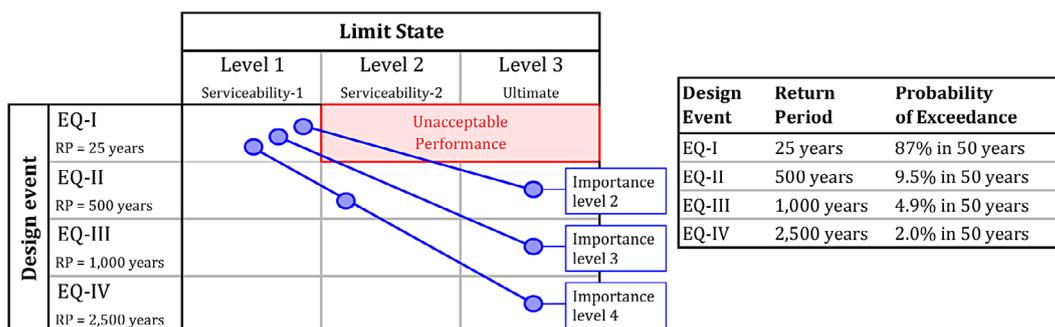


Figure 2. New Zealand seismic performance objectives (for a design life of 50 years).

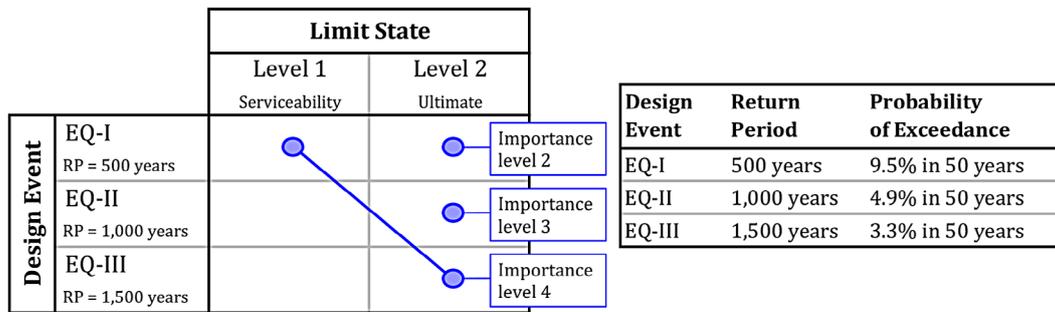


Figure 3. Australian seismic performance objectives.

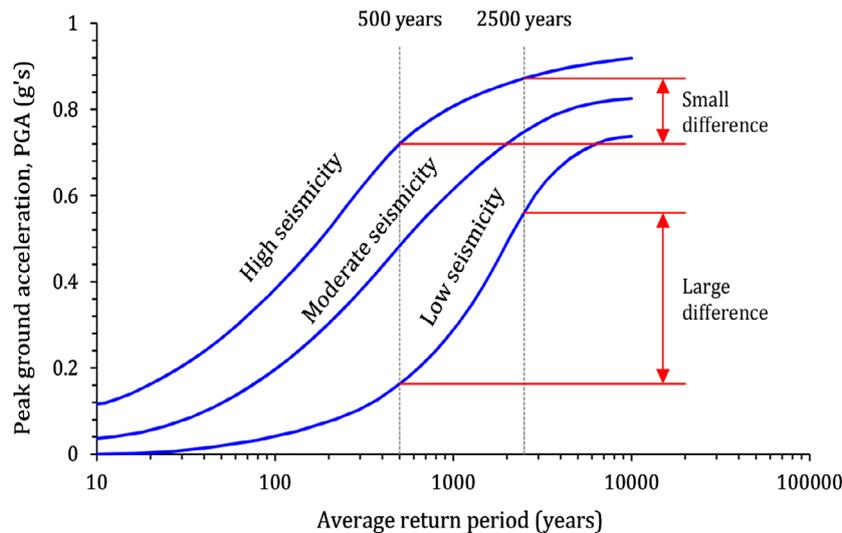


Figure 4. Difference between 500- and 2500-year return period events in regions of low seismicity verse regions of high seismicity (reproduced and modified from Paulay and Priestley (1992)).

of the structure is approached and exceeded, the building develops a rational yielding mechanism that allows maximum energy to be absorbed by the failing structure prior to complete structural collapse, e.g. it aims to force the development of plastic hinges with desirable flexure failure mechanisms at the base of walls or at the ends of beams in moment resisting frames (i.e. strong column weak beam hierarchy). Capacity design originates from New Zealand and has been implemented and become a fundamental part of many design codes around the world, albeit typically limited to regions of higher seismicity. Capacity design was originally the brain child of a New Zealand design engineer John Hollings; however, it was further developed into its current form by prominent earthquake engineering researchers Robert Park and Thomas Paulay (Priestley 2010; Shea 2006). For a detailed discussion on capacity design, the reader is directed to the texts by Park and Paulay (1975) and Paulay and Priestley (1992).

It could be argued that Australia has a similar level of 'conservatism' built into its ULS design procedures such that a collapse prevention performance objective would also be met if a 'real' MCE event were to occur, despite the codes requiring normal building be designed for a mere 500-year RP event. However, there

are a few distinct differences between Australia and New Zealand, in terms of both design philosophy and seismicity. Australian material standards have no mention of capacity design principles nor incorporate them in any respect. It is the opinion of the authors that capacity design was the key ingredient in the New Zealand design philosophy that allowed their buildings to achieve a collapse prevention performance under the much larger than designed level of ground shaking seen during the Christchurch earthquake. Additionally, in regions of lower seismicity like Australia, the difference between the current ULS return period event (e.g. 500-year RP event) and a MCE event (e.g. 2500-year RP event) is much larger than in a region of higher seismicity like New Zealand. This is a widely known phenomenon and has been illustrated in Figure 4.

The Christchurch earthquake was a moderate magnitude intraplate bullseye event, which many Australian seismologists have reported as being the typical type of MCE style event that could occur in any of the major Australian capital cities. It is the opinion of the authors that an event of this nature occurring in any Australian city would cause significant damage and result in an overall building stock performance well below the expectations of the Australian community. Furthermore, partial or complete

structural collapse of buildings, far in excess to what was seen in Christchurch, should be expected in an event of this nature, particularly if located within a soft soil basin.

Based on this, it should be proposed that Australian codes be rectified such that collapse prevention (i.e. ULS performance) is achieved for a much longer RP event. It is noted that in doing this, using current design approaches, the impact to industry would be significant. Current design approaches, which consist of force-based seismic design procedures will inadvertently always under-predict the true ULS capacity of members, meaning current force-based (or strength-based) design methods only allow designers to know a conservative estimate of their structures capacity. By moving away from traditional force-based methods and using displacement-based assessment techniques, in conjunction with the ‘displacement controlled’ phenomenon (Lumantarna, Lam, and Wilson 2013; Lumantarna, Wilson, and Lam 2012), a multi-tier displacement-based assessment procedure could be developed for Australia, which could easily be learnt and adopted by practising engineers. A procedure of this nature would more accurately predict the real behaviour and performance buildings.

The reality is that many buildings in Australia, due to the generally high standard of construction and low seismic nature of the continent, would in fact have sufficient inherent reserve strength to achieve collapse prevention performance under very long RP events (provided good detailing techniques have been adopted). Current force-based design procedures are unable to make this assessment. A displacement-based design procedure would allow for an efficient and more accurate method for checking the seismic compliance of buildings under longer more appropriate RP events. This is an ongoing research objective of the authors.

Any increase of design return periods in the BCA would require across-the-board education to the

profession and industry as to the need and justification for this change. This section aims to initiate this process.

3. AS 1170.4 analysis and design procedure

The seismic design approach in AS 1170.4 has been translated from the conventional force-based approach used in regions of higher seismicity. This approach involves analysing the structure as a linear-elastic system subject to a set of equivalent earthquake design forces, which are taken as the elastic forces divided by a force reduction factor. The elastic forces are the maximum forces the building would be subjected to if the structure were to maintain a linear-elastic response under the associated level of seismic hazard being considered.

The force-based approach is based on the ‘equal displacement phenomenon’ that assumes the inelastic displacement of the structure approximately equals the linear elastic displacement of an equivalent building system, as illustrated in Figure 5. The underlying assumption is that strength is being traded for ductility, i.e. the structure is resisting the applied kinetic energy from the earthquake ground accelerations by a combination of both elastic strain energy and the energy associated with inelastic plastic deformation of the structure – as opposed to purely elastic strain energy in the case of a linear-elastic responding system.

The force reduction factor (R_f) accounts for inelastic behaviour of the building and consists of two components: overstrength (Ω) and displacement ductility (μ). AS 1170.4 uses a structural performance factor (S_p) in lieu of the overstrength factor, which is equal to the reciprocal of overstrength, i.e. $1/\Omega$. Resulting in the force reduction factor used in AS 1170.4 being taken as $R_f = \mu/S_p$, which is equivalent to $R_f = \Omega\mu$.

The force reduction factor means the performance of the building during the design earthquake event is

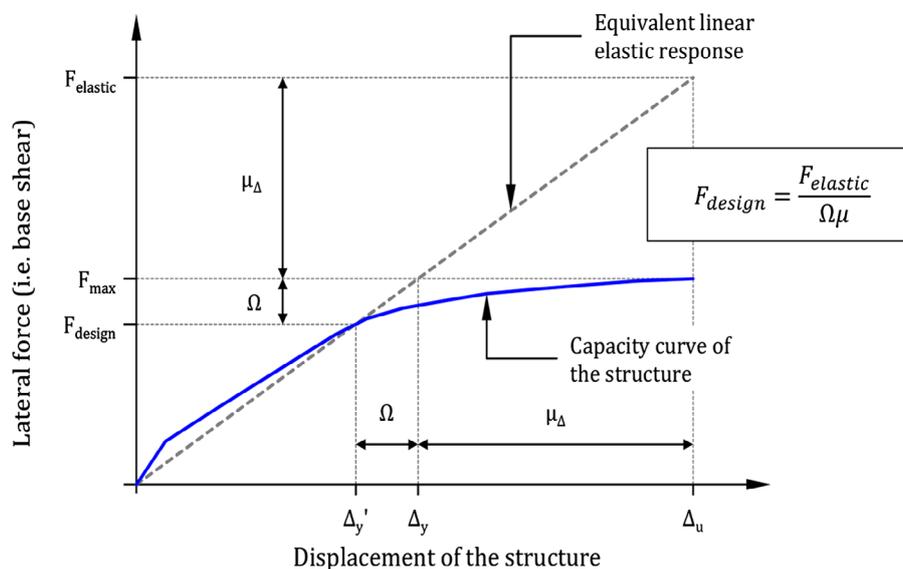


Figure 5. Idealisation of the force-based seismic design methodology.

not equal to the displacement corresponding to that of the elastic design model of the building, but rather to a point equal to $\Omega\mu$ times the response of the elastic design model. This is one of the primary differences between wind actions and seismic actions and in part why the commonly held belief that if the base shear resulting from wind actions exceeds the base shear from an equivalent static seismic analysis, seismic actions can simply be ignored, is inappropriate and false. It is imperative that the designer details the structure to ensure it has the level of ductility assumed in the seismic design calculations, irrespective of how large the wind actions are.

AS 1170.4 requires different levels of analysis depending on the ‘earthquake design category’ (EDC) of the building. The EDC is a function of the Importance Level of the building, the seismic hazard, the sub-soil class of the site where the building is located and the overall height of the building. EDC I requires an equivalent static analysis where the lateral force at each storey is equal to 10% of the seismic weight for that respective storey; EDC II requires an equivalent static analysis where the lateral force is determined using pseudo dynamic properties of the building determined from empirical formulas and an appropriate response spectrum for the site the building is located on; while with EDC III, a multi modal dynamic analysis is required.

AS 1170.4 will typically require designers to perform either an equivalent static analysis or multi modal dynamic analysis of the structure, both of which are force-based analysis methods. The latest version of AS 1170.4 allows designers to opt for a displacement-based assessment method, however further research is required before this could suitably be applied to complex RC building systems. The authors, however, have presented a ‘first cut’ on how displacement-based methods could possibly be performed in accordance with AS 3600 and AS 1170.4 in Menegon et al. (2015a).

4. Ductility and overstrength factors

Ductility and overstrength are fundamental factors in force-based seismic design. The adoption of detailing practices that allow the structure to develop the amount of inelastic behaviour, which is assumed to a varying degree depending on which factors are selected, are essential to ensure the structure performs as intended. The ductility and overstrength factors for RC structures can either be taken from the tables provided in AS 1170.4 (Standards Australia 2007) or from a non-linear static pushover analysis of the structure.

There are essentially three ductility classes to choose from for RC structures in the tables provided in AS

1170.4: ‘limited ductile’, where $\mu = 2.0$ and $\Omega = 1.3$; ‘moderately ductile’, where $\mu = 3.0$ and $\Omega = 1.5$; and ‘fully ductile’, where $\mu = 4.0$ and $\Omega = 1.5$. The first ductility class, limited ductile, can be adopted when the structure is detailed in accordance with the main body of AS 3600 (Standards Australia 2009). The second ductility class, moderately ductile, can be adopted when the structure is detailed in accordance with the main body of AS 3600 and Appendix C of AS 3600. The third ductility class, fully ductile, can only be adopted when the structure is designed and detailed in accordance with the New Zealand earthquake loading code, NZS 1170.5 (Standards New Zealand 2004) and New Zealand concrete standard, NZS 3101 (Standards New Zealand 2006), which includes the systematic adoption of capacity design principles and specifying all reinforcement to be E grade to AS/NZS 4671 (Standards Australia and Standards New Zealand 2001). It should be noted that E grade reinforcement is generally not available in the Australian market and typically requires a special order be made by the supplier. The three ductility classes for RC design in Australia are summarised in Table 1.

Designers are recommended to generally adopt the limited ductile classification and not blindly adopt higher classifications, such as moderately ductile. One should be well versed with the theory and application of earthquake engineering if the higher moderately ductile classification is used. The moderately ductile classification has a force reduction factor of 4.5, which is 73% higher than the limited ductile classification that has a force reduction factor of just 2.6. This significantly higher force reduction factor does not ‘come for free’ and requires the associated higher level of detailing to ensure this extra ductility and inelastic behaviour can actually be achieved in the structure.

The seismic design Appendix C in AS 3600 is long overdue to be updated and in its current form, does not provide adequate direction for the detailing of RC walls to achieve such a high level of ductility (i.e. $\mu = 3$). It is recommended to all designers who wish to adopt a structure classification of moderately ductile, i.e. $\mu = 3$, to generally follow the guidance provided by NZS 3101 for a ‘ductile structure’ and perform a non-linear static pushover analysis to ensure their structure has this level of ductility.

For certain detailing practices, a fourth ductility class is being recommended: ‘non-ductile’, where $\mu = 1.0$ and $\Omega = 1.3$. This ductility class would include RC walls detailed with either or a combination of low ductile reinforcement, central layers of vertical reinforcement or minimal percentages of vertical reinforcement (i.e.

Table 1. Ductility classes for RC design in Australia.

Ductility class	μ	Ω	R_f	Notes
Limited ductile	2.0	1.3	2.6	The structure is detailed in accordance with the mainbody of AS 3600
Moderately ductile	3.0	1.5	4.5	The structure is detailed in accordance with the main body and Appendix C of AS 3600
Fully ductile	4.0	1.5	6.0	The structure is designed and detailed in accordance with NZS 1170.5 and NZS3101

less than about 0.7%); which are all detailing practices that extremely hinder a wall's ability to develop ductility, yet are extensively being used in low and mid-rise buildings around Australia. This non-ductile class of walls is discussed further below. It is worth noting that AS 3600 is currently undergoing a major revision with respect to seismic design and the current draft that has been released for public comment, proposes to include this fourth ductility class.

5. Reinforced concrete walls

The majority of low, mid and high-rise multi-storey buildings in Australia use RC walls as the major lateral load resisting system of the structure. Traditionally this consists of (i) a system of isolated cast *in situ* RC walls, (ii) a central or eccentric cast *in situ* RC building core, or (iii) a combination of both former options. In recent years, the adoption of precast concrete walls and jointed precast building cores over traditional cast *in situ* RC elements has become increasingly popular in low and mid-rise construction, particularly in the south-eastern Australian states.

Precast walls are usually cast off site as individual rectangular walls and erected on site as individual walls or connected together to form jointed precast building cores. The top and bottom of the wall is connected to foundations or walls above/below using dowel connections and the sides of the wall are connected to adjacent walls (if present), using welded stitch plate connections. Precast walls offer many advantages for contractors in the form of significant time and cost savings, which has significantly contributed to their rapid adoption in recent years. In conjunction with this rapid rise in the popularity in recent years, has come the adoption of some derelict detailing practices.

The detailing of RC walls in Australia is discussed further and more comprehensively in a previous publication by the authors (Menegon et al. 2017), which reviewed 31 case study buildings and documented commonly observed detailing practices for both cast *in situ* and precast RC walls.

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

- Lateral forces to be distributed based on individual wall stiffness's determined from gross cross-sectional properties. At a minimum, forces should be distributed using the cracked section properties of the walls. The reader is directed to Priestley, Calvi, and Kowalsky (2007) for a detailed discussion on the matter.
- Designers can neglect the requirement for restraining vertical reinforcement in walls when they are designed as columns. Walls subjected to inelastic in-plane deformations, which is assumed when force reduction factors greater than 1 are taken, would likely undergo concrete compressive strains in their extreme compressive fibre much higher than that of columns under ultimate gravity load conditions which must adhere to this requirement.

5.1. RC walls detailed with a single central layer of reinforcement

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

Precast walls in multi-storey buildings are, however, very commonly detailed using one central layer of vertical and horizontal reinforcement. RC walls detailed with one central grid of vertical and horizontal reinforcement can have relatively poor performance – compared to walls detailed with two grids of reinforcement – under cyclic in-plane loading seen during an earthquake. The end regions of walls subjected to cyclic in-plane loading are essentially subjected to cyclic tension-compression axial loading. The lateral stability of walls with one central grid of reinforcement is significantly lower than walls with two grids of reinforcement when this type of loading is present. This has been shown in experimental laboratory testing (e.g. Menegon et al. (2015b); Rosso et al. (2017a); Rosso, Almeida, and Beyer (2017b)) and recent earthquakes, with one notable example being the Pyne Gould Corporation Building which collapsed during the 2011 Christchurch Earthquake. The Pyne Gould Corporation Building was a four storey RC building that had a large central building core that was detailed with one central grid of vertical and horizontal reinforcement.

Section 11 *Design of Walls* in AS 3600 sets out the design procedure and detailing requirements for RC

walls. Clause 11.7.3 restricts the use of centrally reinforced RC walls in a number of different scenarios; an extract from the clause is reproduced below:

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

- (a) Walls greater than 200 mm thick.
- (b) Any part of a wall structure where tension exceeds the tensile capacity of the concrete under the design ultimate loads.
- (c) Walls designed for two-way buckling (based on Clauses 11.4(b) or 11.4(c)).

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

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

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

It should be noted that in some applications of AS 3600 the aforementioned clause (i.e. 11.7.3) do not need to be adhered to when detailing walls. However, in these instances walls are required to be designed and detailed as columns which are required to that have bars on each face with closed ligatures – i.e. detailing requirements far in excess of what has been discussed.

5.2. RC walls detailed with low ductility reinforcement

AS 3600 generally requires designers to specify either N grade (i.e. normal ductility) or L grade (i.e. low ductility) reinforcement manufactured in accordance with AS/NZS 4671 (Standards Australia and Standards New Zealand 2001). N grade reinforcement comes in the form of grade D500N bars (i.e. deformed straight or coiled rebar with a minimum characteristic yield stress of 500 MPa) and L grade reinforcement comes in the form of grade D500L welded mesh (i.e. deformed bars welded together as a mesh with a minimum characteristic yield stress of 500 MPa). The major difference between D500N and D500L is the minimum characteristic ultimate strain of 5 and 1.5% respectively and the strain hardening ratio (i.e. the bars ultimate stress to yield stress ratio) of 1.08 and 1.03 respectively. AS 3600 generally requires a 0.8 capacity reduction factor for N grade reinforcement and 0.6 capacity reduction factor for L grade reinforcement.

Quite commonly detailing of precast concrete walls consists of either a central layer or two layers, one per face, of L grade mesh, with additional N grade vertical bars as required (refer Figure 6). A common design approach when calculating the capacity of RC walls with L grade mesh is to reduce the area of steel provided by the mesh by a factor of 0.8 to account for the more severe reduction factor of L grade mesh compared to N grade rebar required by AS 3600. No consideration is given to the reduced ultimate strain of the mesh. This approach originates from the Steel Reinforcement Institute of Australia (SRIA) design guide for RC slabs detailed with L grade mesh (Steel Reinforcement Institute of Australia 2008). While this approach may be appropriate for RC slabs as outlined in the design guide, its translation to RC walls is inappropriate, especially in respect to the seismic design of RC walls where inelastic behaviour (i.e. plastic deformation) of the wall is assumed as part of the analysis procedure.

When designing RC walls for seismic actions, using force-based procedures where explicit assumptions regarding ductility are made, it is acknowledged that the lateral load resisting elements will undergo inelastic plastic deformations. This can only happen after the cracking moment capacity of the wall has been exceeded and the tension strains are transferred and resisted by

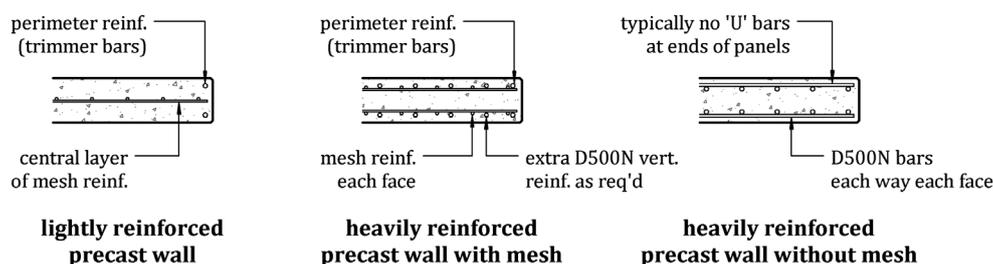


Figure 6. Typical plan sections of precast wall (Menegon et al. 2017).

the vertical reinforcement. The vertical reinforcement then begins to yield and strain harden as the ductility in the wall is developed. The reduced ductility and ultimate strain of L grade mesh extremely inhibits the wall's ability to develop ductility. This is illustrated in Figure 7, which shows a comparison of the moment-curvature response of a typical building core using both D500N and D500L material properties in the analysis. The curvature ductility of the section is significantly reduced when low ductility reinforcement is adopted. The response was calculated using a fibre-element model as outlined in Menegon et al. (2015a). Each respective moment-curvature response curve highlights where the 'characteristic ultimate strain' would (theoretically) be reached and where the 'mean ultimate strain' of the reinforcement is reached. The reader should note the mean ultimate strain of D500N and D500L reinforcement has been determined to be approximately 3.3 and 9.5%, respectively (Menegon et al. 2015c).

It has been shown in experimental testing of RC slabs that when L grade reinforcement is adopted the ductility is significantly limited or eliminated completely. This was highlighted by Gilbert and Sakka (2007) where they showed that while RC slabs reinforced with low ductility mesh could 'easily meet their strength requirements, the slabs lacked robustness, with little ability to deform plastically'.

Typical stress-strain curves for N grade and L grade reinforcement samples are shown in Figure 11, highlighting the reduced ductility of L grade mesh. Additionally, the significantly reduced strain hardening ratio of low ductility reinforcement inhibits the development of a distributed plastic hinge mechanism as it doesn't allow for the plastic tension strains to be effectively distributed vertically across the plastic hinge region of a wall.

Therefore, similar to the previous example, designers, when detailing walls using low ductility reinforcement, should adopt the proposed non-ductile classification

with a displacement ductility factor of 1.0 when determining the seismic actions.

5.3. Lightly reinforced RC walls

AS 3600 allows RC walls to be detailed with very small percentages of reinforcement, where the lower limit for vertical and horizontal reinforcement is 0.15 and 0.25%, respectively. These percentages have been selected with respect to thermal restraint and crack control requirements and have generally been observed to be sufficient for those purposes. However, when the vertical reinforcement in the end region of a wall is below a certain limit, distributed cracking cannot be developed in the wall resulting in the development of one or two major cracks with concentrated plasticity in these locations. This behaviour greatly effects a wall's ability to develop ductility. Desirable distributed cracking with distributed plasticity allows for larger in-plane displacements and ductility values to be developed as it limits the local tension strain in the vertical reinforcement, which can be a limiting factor for the displacement capacity for an RC wall. This behaviour is illustrated indicatively in Figure 8 where two walls have equal magnitudes of lateral displacement, yet the maximum plastic tension strain in the extreme tensile reinforcement for the wall with distributed cracking (Figure 8(a)) is significantly smaller than the wall with an undesirable single crack (Figure 8(b)), as the latter has all the inelastic behaviour concentrated in one location (i.e. the single crack at the base of the wall).

The minimum percentage of vertical reinforcement required in a wall to ensure desirable distributed cracking occurs has received much attention in recent years, particularly following the 2011 Christchurch earthquake, where this problem was commonly observed in RC walls (Henry 2013). Lu et al. (2017) and Lu and Henry (2017) performed a comprehensive investigation, consisting of both experimental laboratory testing and

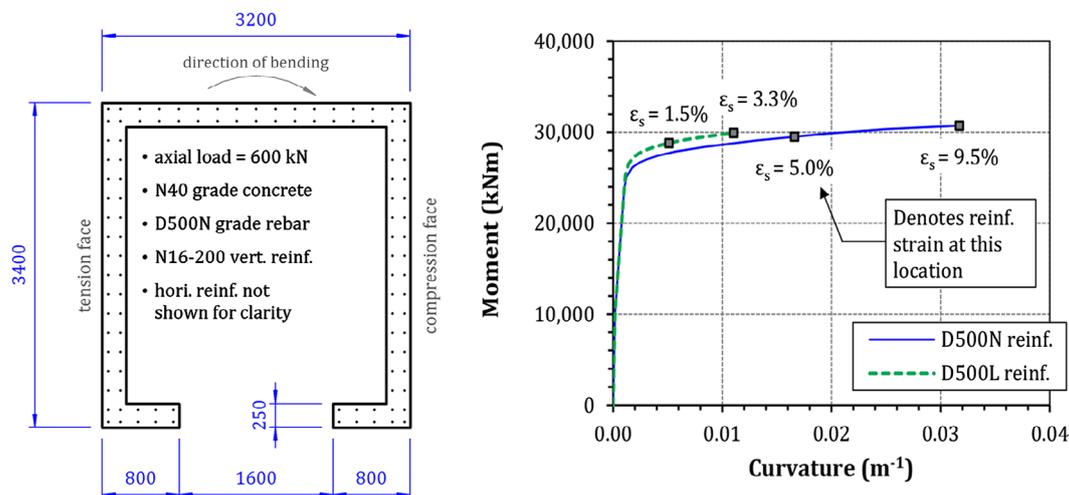


Figure 7. Moment-curvature response of a building core – comparison of D500N and D500L rebar.

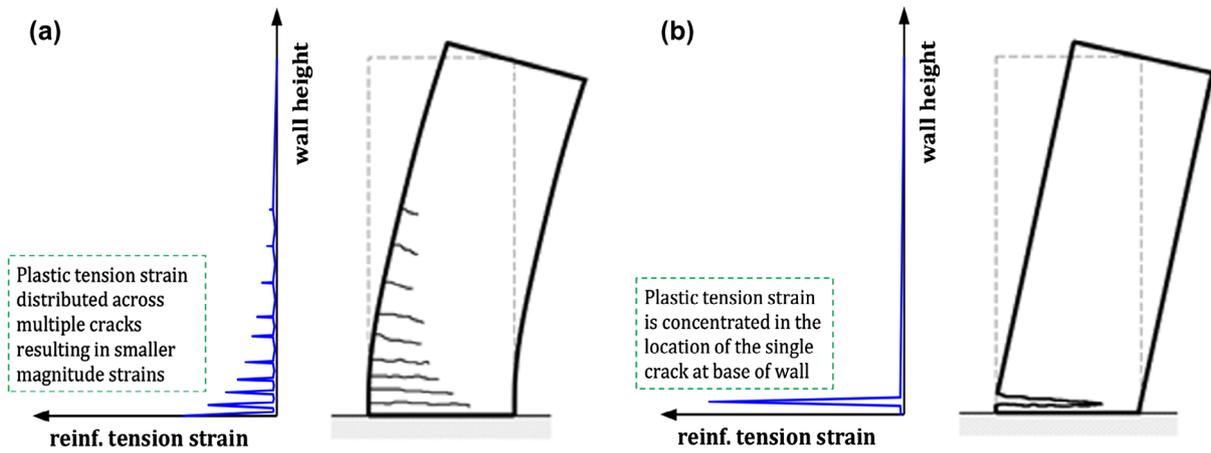


Figure 8. Strain distribution of the extreme tensile reinforcement in an RC wall for different crack distributions. (a) RC wall with distributed cracking. (b) RC wall with single crack.

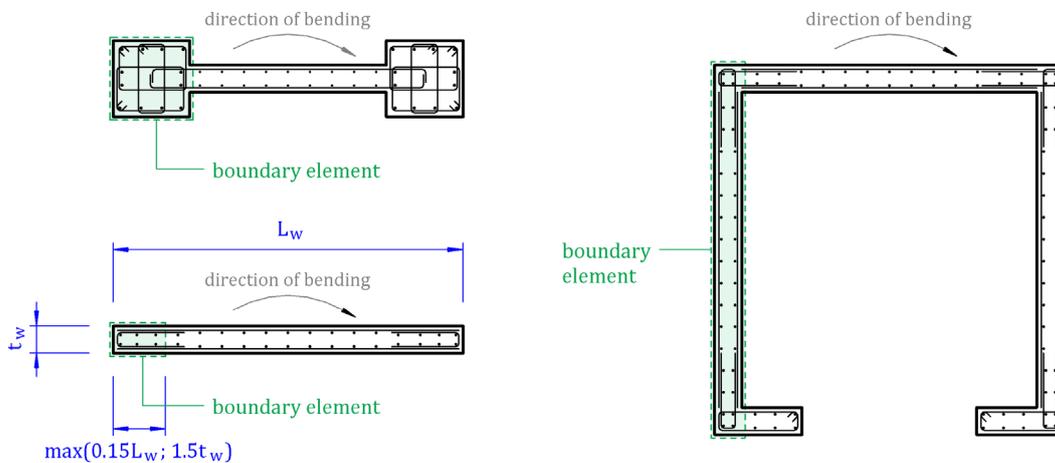


Figure 9. Boundary elements of various wall cross sections. The boundary element of a rectangular wall is defined in accordance with Eurocode 2 (European Committee for Standardization (Cen) 2004).

finite element computer modelling, which highlighted the poor displacement behaviour that results when an RC wall cannot develop distributed cracking. This behaviour was also identified and reported by Hoult, Goldsworthy, and Lumantarna (2017) in an independent study using calibrated finite element computer models of RC walls.

This minimum percentage of vertical reinforcement required to ensure distributed cracking can quickly be determined intuitively by examining the tensile strength of the concrete in the end region of the wall (i.e. the boundary element of the wall, refer Figure 9) versus the amount and strength of the reinforcement in that respective area. If the tensile strength of the concrete in that region (i.e. $A_c f_t$) is weaker than the yield strength of the reinforcement (i.e. $A_{st} f_{sy}$), distributed cracking will occur. Equation 1 can be derived using this logic:

$$\begin{aligned}
 A_{sy} f_{sy} &\geq A_c f_t \\
 p_v f_{sy} &\geq f_t \\
 p_v &\geq \frac{f_t}{f_{sy}} \\
 p_{v.min} &= \frac{f_t}{f_{sy}}
 \end{aligned} \tag{1}$$

The ultimate tensile strength of concrete is usually expressed as either the direct tensile strength or the flexural tensile strength. The latter relates to members where the tensile stresses are due to bending/flexure actions, where concrete can sustain a somewhat higher level of tensile stress due to the varying strain gradient across the depth of the section. This is typically the case for sections that are not exceedingly deep. As the section depth (i.e. wall length, L_w) increases the flexural tensile strength approaches and eventually equals the direct tensile strength. The Eurocode 2 (European Committee for

Standardization (Cen) 2004) model proposes this occurs when the section depth is 1600 mm or greater. It would be unusual for walls to have a length less than 1600 mm and as such, for a generalised approach, the authors are recommending the direct tensile strength be used.

AS 3600 specifies the ultimate direct tensile strength of concrete to be 0.36 multiplied by the square root of the characteristic compressive strength, i.e. $0.36 \sqrt{f'_c}$. The code then suggests a factor of 1.8 for the ratio of lower to upper characteristic tensile strength. When calculating the minimum percentage of reinforcement to ensure distributed cracking using Equation 1, the authors are recommending to use the upper characteristic tensile strength of concrete given the high variability in concrete mix designs seen on site and the importance of achieving distributed cracking in an RC wall, i.e. $f_t = 1.8 \times 0.36 \sqrt{f'_c}$. For typical 500 MPa reinforcement, the minimum percentages of vertical reinforcement required to achieve desirable distributed cracking for various standard grades of concrete are summarised in Table 2.

Similar to the first two scenarios, the designer should adopt the proposed non-ductile classification with a displacement ductility factor of 1.0 when determining the seismic actions if the walls, in the plastic hinge regions, have vertical reinforcement percentages lower than the values presented in Table 2.

5.4. Detailing of precast wall dowel connections

The typical detail for connecting precast concrete wall panels vertically in multi-storey buildings is a grouted dowel connection. This consists of a corrugated grout tube cast into the bottom of the top panel, which when erected, slots over vertical bars cast into the element below. The corrugated grout tubes are then grouted after erection, typically using cementitious grout with a characteristic compressive strength 10 to 20 MPa stronger than the respective panel strength. This detail is shown in Figure 10.

This detail is most commonly constructed using N grade reinforcing bars as dowels. While not a common practice, it is occasionally observed that designers specify all-thread for the dowels. All-thread, most commonly, comes in two grades, 4.6 (i.e. mild steel) and 8.8 (i.e. high tensile). Grade 4.6 and 8.8 all-thread have a minimum characteristic ultimate tensile stress of 400 and 830 MPa respectively and a yield stress equal to approximately 60 and 80% respectively of their respective ultimate tensile stress. The mild stress grade 4.6 all thread has a much

higher ultimate strain compared to that of high tensile grade 8.8 all-thread, as shown in Figure 12.

Despite the mild steel grade 4.6 all-thread having a very high ultimate strain and strain hardening ratio (i.e. ultimate stress to yield stress ratio) – the two generic factors required for reinforcement in ductile RC elements – it is still ill-suited for using as reinforcement or dowels in RC elements. This is due to the majority of strain hardening occurring over a very small region from about 1.5 to 2.5%, with the remaining much smaller proportion occurring from 2.5 to 17% (refer to Figure 11). This is in stark contrast to D500N grade reinforcement, which has the strain hardening distributed over a much larger range (e.g. from about 2 to 10% in Figure 11). Similarly, high tensile grade 8.8 all-thread is also ill suited for using as reinforcement or dowels in RC elements since it has a much smaller ultimate strain compared to that of D500N grade reinforcement. Recommended best practice for detailing dowel connections in RC walls is to specify D500N grade dowels only.

As discussed previously, when ductility is assumed, it is acknowledged that the wall will undergo tension forces in the end regions (irrespective of what your analysis shows using the reduced seismic loads as discussed with reference to Figure 5). Therefore, when ductility is assumed in the design, the total area of dowel bar steel at the base of the wall has to be equal to or greater than the area of vertical steel provided in the wall. If not, an adequate load path that is consistent with the assumptions made in the analysis process regarding ductility has not been provided. In summary, if the dowels in precast construction are specified to be grade 4.6 or 8.8 all-thread and or specified as D500N bars but having a total area of steel less than the vertical reinforcement in the wall, then proposed the non-ductile classification with a ductility factor of 1.0 should be adopted.

5.5. Overall system behaviour and precast walls as gravity load carrying elements only

Multi-storey buildings, particularly apartment buildings, are commonly constructed using a central or eccentric cast *in situ* building core and rectangular RC precast walls around the perimeter of the building. These perimeter walls typically support the floor slab and are often referred to as 'gravity load carrying elements only' and detailed with a central layer of L grade reinforcing mesh. The cast *in situ* core is assumed to take all the lateral load. However, these perimeter walls are tied into the floor diaphragm and as such will take lateral load during a seismic event. Therefore, consideration of the different points discussed above in Sections 5.1–5.4 is still required. It would be a false assumption to assume these walls are gravity load carrying only elements, as an earthquake will distribute lateral load to all elements linked by the floor or roof diaphragm based on their respective in-plane strength and stiffness,

Table 2. Minimum boundary element reinforcement ratio to ensure distributed cracking in RC walls.

	Concrete grade: N32	Concrete grade: N40	Concrete grade: N50	Concrete grade: S65
$p_{v,min}$ =	0.7%	0.8%	0.9%	1.0%

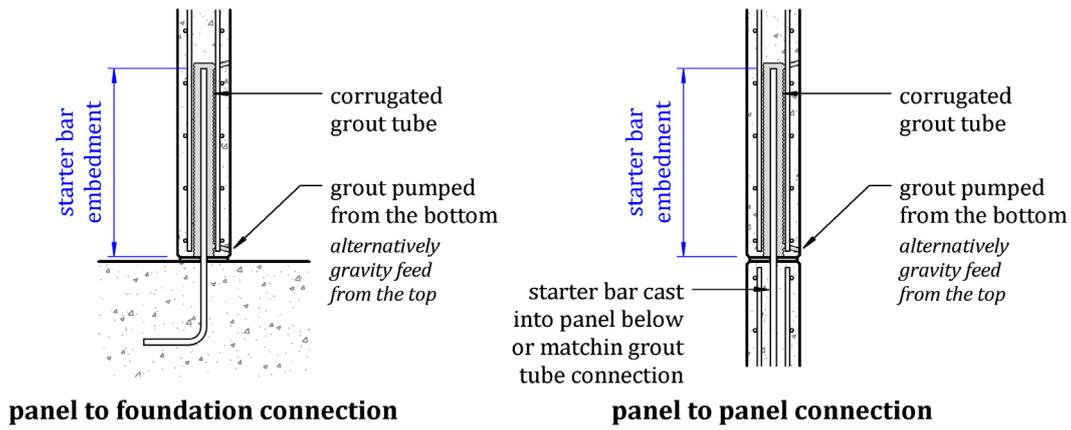


Figure 10. Grout tube connection (Menegon et al. 2017).

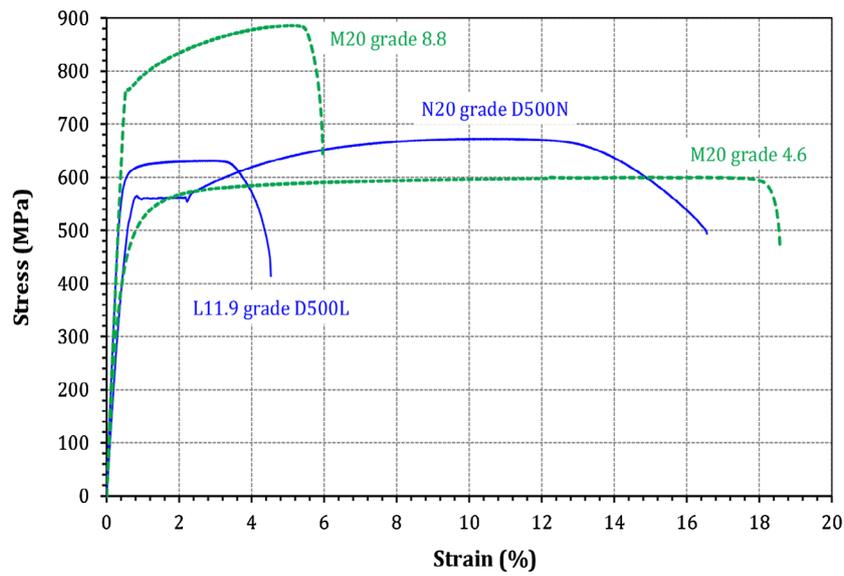


Figure 11. Typical stress–strain curves of various reinforcement and all thread grades.

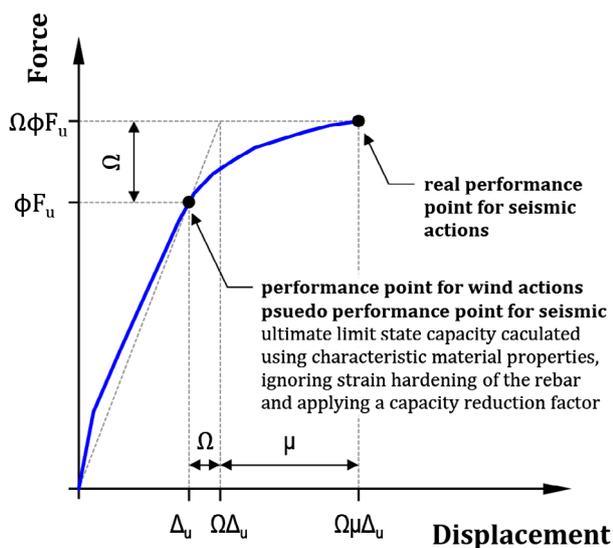


Figure 12. The ‘real’ performance point for seismic actions.

regardless of their common held description. Behaviour of the overall system must be considered. Further, this is explicitly required in AS 1170.4 where it is noted in

Clause 5.2.3 that these elements must be ‘considered to be part of the seismic-force-resisting system and designed accordingly’.

Often the lateral load is distributed between elements (e.g. walls and or cores) based on the initial gross second moment of inertia value of the RC element. If the seismic loads are determined using a ductility factor of 2.0 (or higher) and it is shown that these walls take minimal lateral load, it would still be false to ignore the considerations discussed in Sections 5.1–5.4. This approach would not account for the reduced stiffness of the core after (a) the cracking moment capacity of the core is exceeded and (b) the inelastic plastic deformation required to develop the ductility factor of 2.0. The overall behaviour of the building must be considered to ensure displacement capability of the overall structure is achieved. The lateral load should be distributed based on the effective (i.e. first yield or ‘cracked’) section properties of each element and it should be emphasised again, that the actual displacement or performance point of the structure is equal to a point equal to $\Omega\mu$ times the response of the

elastic design model (as highlighted in Figure 12). All elements in the structure must be designed and detailed such that they perform adequately when subjected to this amount of lateral displacement.

Torsional effects must also be considering when the overall system behaviour of a building or structure is being assessed. Torsional raking effects will be induced in the building if the centre-of-stiffness and centre-of-mass of the building do not align during seismic actions. Due to the inherent uncertainties in calculating the centre-of-mass AS 1170.4 requires the earthquake actions to be applied at a position that is either $\pm 0.1b$ from the calculated centre-of-mass, whichever gives the worse effect, where b is equal to plan dimension of the structure perpendicular to the direction of loading being considered. Designers should attempt to minimise the distance between the centre-of-stiffness and centre-of-mass of the building to reduce the possible amount of torsional behaviour under seismic actions.

5.6. Axial load ratio

The simplest and most effective method for increasing the displacement capacity of RC elements is to reduce the axial load ratio of the column or wall. The axial load ratio is defined as the axial load on the element divided by the product of the gross cross sectional area and ultimate compressive strength of the concrete column or wall. This was highlighted in a recent study by Wilson et al. (2015), where a simplified model for predicting the maximum drift capacity of RC columns was developed. An example has been provided in Figure 13 to illustrate this behaviour. In this example, the lateral drift at axial load failure (i.e. complete structural collapse) is reduced by a factor of 2.3 when the axial load ratio is increased from 0.10 to 0.30.

Alarcon, Hube, and Liera (2014) performed an experimental study where three identical walls with different axial load ratios were tested under quasi-static cyclic lateral load until failure. Axial load ratios of 0.15, 0.25 and 0.35 were used for the three specimens and the

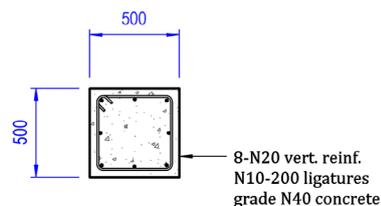


Figure 13. The effect of axial load on RC columns.

specimens failed at 2.7, 1.8 and 1.5% drift respectively. In this study, it was shown that the drift capacity of the wall decreased by a factor of 1.8 when the axial load ratio is increased from 0.15 to 0.35, which is in good agreement with the study by Wilson et al. (2015) and Figure 13.

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

6. Summary of recommendations

The recommendations made throughout this paper have been summarised for the convenience of the reader in Table 3. Additional notes are also provided to assist in implementing these recommendations into design scenarios. Where reference to NZS 3101 is made, the intent is that the recently released amendment 3 version of the code is adopted. Additional items, outside the scope of this paper, which designers should consider when performing seismic analyses of RC wall buildings include:

- Slenderness ratios of walls, i.e. height-to-thickness ratios, to prevent out-of-plane instabilities under reversed lateral load, as discussed in Menegon et al. (2015b), Rosso, Almeida, and Beyer (2016) or Dashti, Dhakal, and Pampanin (2017). These studies are ongoing research efforts and do not currently provide firm guidance or design rules for designers. In the meantime, Wallace (2012) has recommended limiting the h/t_w ratio to 16 or less, where h is the inter-storey unrestrained height of the wall. Wallace (2012) also precautions that a value of 16 may still be insufficient to prevent out-of-plane instabilities in some wall configurations, however, until more comprehensive studies are completed, a h/t_w limit of 16 is being recommended for plastic hinge regions or elsewhere in RC walls where significant inelastic plastic tension strains can be developed.

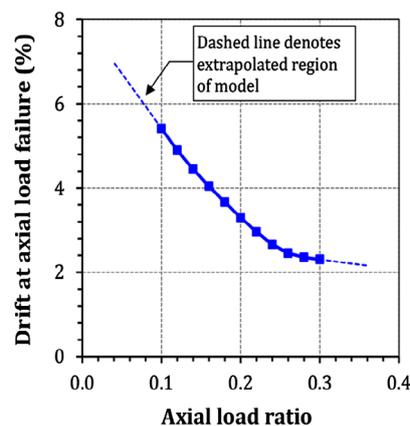


Table 3. Summary of recommendations.

	Non-ductile	Limited ductile	Moderately ductile
Ductility factor, μ	1	2	3
Overstrength factor, Ω	1.3	1.3	1.5
Structural performance factor, S_p	0.77	0.77	0.67
Force reduction factor, R_f	1.3	2.6	4.5
Low ductility reinforcement	Permitted	Not permitted	Not permitted
Centrally reinforced walls	Permitted	Not permitted	Not permitted
Vertical reinforcement ratio in plastic hinge regions of the wall ^a	Current code provisions	As given in Table 2 ^b	As given in Table 2 ^b
Confinement (cross tie) reinforcement	Current code provisions	Current code provisions	NZS 3101 provisions ^c
Splicing of vertical reinforcement in plastic hinge regions of the wall ^a	Lap splices permitted	Lap splices permitted	NZS 3101 provisions ^c or mechanical splices ^d
Development of horizontal reinforcement ^e	No requirement	Required at ends of walls and all wall intersections	Required at ends of walls and all wall intersections
Axial load ratio ^f	≤ 0.20	≤ 0.20	≤ 0.20
Shear design	No requirement	$\phi V_u \geq (1.6M_u/M^*)V^*$	$\phi V_u \geq (1.6M_u/M^*)V^*$
Slenderness ratio in plastic hinge regions of the wall ^a	Current code provisions	$\frac{h}{t_w} \leq 16$	$\frac{h}{t_w} \leq 16$
Aspect ratio ^g	No requirement	$\frac{H_w}{L_w} \geq 2$	$\frac{H_w}{L_w} \geq 2$
Precast construction	Permitted	Permitted	Permitted
Dowel bars in precast grout tube connections	No requirement	D500N bars only	D500N bars only
Dowel area in precast grout tube connections	No requirement	$A_{st,dowel} \geq A_{st,wall}$	$A_{st,dowel} \geq A_{st,wall}$

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

^bThe vertical reinforcement ratio can be decreased at a rate of 15% per storey away from the plastic hinge regions.

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

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

^ee.g. additional 'U' bars lapped with horizontal reinforcement at wall intersections and end regions of walls or cogging horizontal reinforcement into confined end regions of walls (refer Menegon et al. (2017)).

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

^gWalls with an aspect ratio less than 2 should be designed as a non-ductile element using strut and tie methods.

- Ensuring a desirable flexure failure occurs and not an undesirable brittle shear failure, i.e. shear behaviour of the wall should not be allowed to control the strength of the wall. This could be achieved by designing for a minimum shear force value that corresponds to the actual flexural moment capacity of the wall, i.e. multiply the design shear force by a value corresponding to the actual moment capacity of the wall divided by the design moment acting on the wall. Priestley, Calvi, and Kowalsky (2007) proposes that the overstrength for an RC element, where the capacity has been calculated using characteristic material properties and no allowance for strain-hardening in the reinforcement, can be taken as 1.6. Therefore, $\phi V_u \geq (1.6M_u/M^*) \times V^*$.

7. Conclusions

This paper has examined and discussed the seismic design procedure in Australia for RC wall buildings using AS 1170.4 and AS 3600. The key points of the paper are summarised as follows:

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

2. A general misconception by designers is that the lateral stability under wind load – which in many scenarios is the predominant environment load for buildings in Australia – envelops any seismic design requirement and such an attitude could lead to substantial risks under a seismic event. It is imperative that RC buildings are detailed such that they can achieve the level of ductility assumed in the design, irrespective of what the wind actions are.
3. Seismic analysis assumes strength is being traded for ductility through the use of the force reduction factor, which is a combination of a displacement ductility factor and structural performance (or overstrength) factor. The designer needs to directly consider how the structure can develop the amount of ductility being assumed and what effect this can have on other structural elements in the building.
4. Designers should adopt the ‘limited ductile’ structure classification to AS 1170.4 and AS 3600, which allows for a displacement ductility and overstrength factor of 2.0 and 1.3 to be used respectively. The ‘moderately ductile’ classification should only be used when the more onerous detailing approaches recommended within the paper are adopted. Adopting larger ductility and overstrength factors and hence, larger force reduction factors, does not come for free and requires the associated level of detailing to ensure the extra ductility is achieved.
5. Designers should adopt a non-ductile structure classification and assume a displacement ductility factor of 1.0 when detailing RC walls with low ductility reinforcement, central layers of vertical reinforcement or low percentages of vertical reinforcement (i.e. less than 0.7%). In many scenarios when ductility is assumed in the seismic design process, walls detailed with single central layer of vertical reinforcement do not comply with Section 11 of AS 3600 and as such, are not code compliant.
6. The overall system behaviour of a building must be considered, including any torsional effects. Precast walls around the perimeter of a building (often referred to as load bearing only elements) cannot be assumed to take zero lateral load by assuming the central core takes 100% of the lateral load. While it may appear that these perimeter walls take minimal lateral load when distributing elastic forces based on initial or effective stiffnesses, the real forces could be somewhat different and larger when the overall system’s inelastic displacement is taken into consideration. When performing seismic analysis, which trades strength for ductility, displacement compatibility of the whole

system needs to be thoroughly considered and established.

7. The paper is concluded by presenting a summary table of the recommendations made throughout the paper for ease of the reader and implementation in a design office scenario.

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Peter McBean is the joint managing director of Wallbridge Gilbert Aztec Consulting Engineers and has over 30 years' experience as a consulting structural engineer. Career highlights include his role as Structural Engineering Design Director for the new \$1.85b Royal Adelaide Hospital. He is a fellow of The Institution of Engineers Australia, a Chartered Professional Engineer and the current National President of the Australian Earthquake Engineering Society. Peter is a member of BD6/11 and BD2, the standards committees responsible for AS 1170.4 and AS 3600 respectively. Peter co-authored the recently updated SRIA publication titled Guide to Seismic Design and Detailing of Reinforced Concrete Buildings in Australia. In addition to design activities, Peter is an active Australian Urban Search and Rescue (USAR) Task Force Engineer. This role took him to Christchurch following the devastating earthquake of February 2011 and he was awarded the Humanitarian Overseas Service Medal by the Australian Government for his services in Christchurch.

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