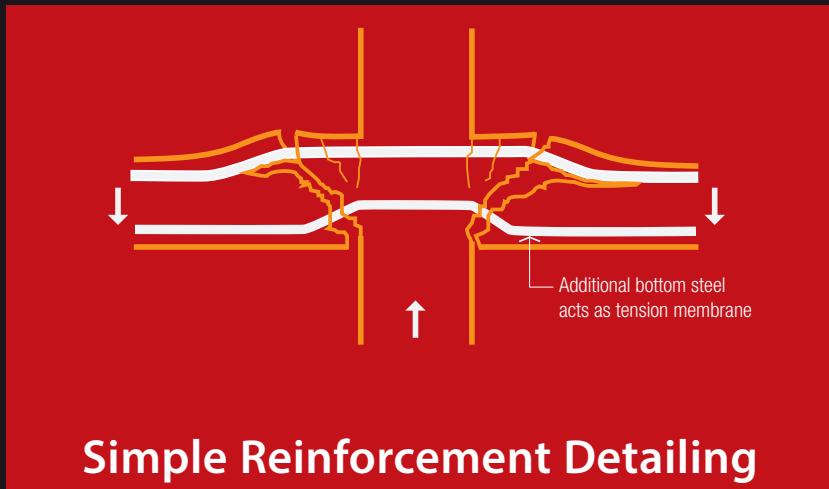


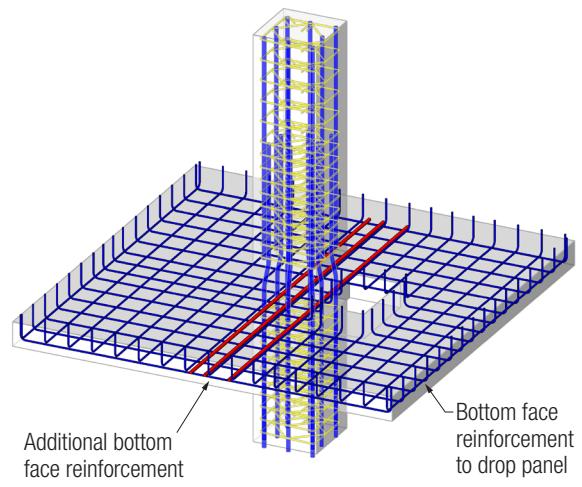
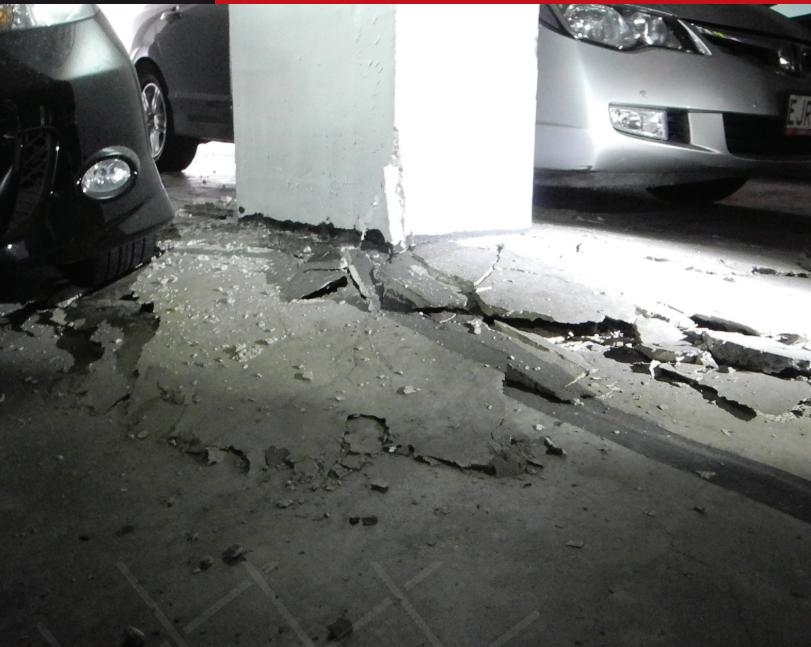
Guide to Seismic Design and Detailing of Reinforced Concrete Buildings in Australia



Simple Reinforcement Detailing



Improves Life Safety



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Steel Reinforcement
Institute of Australia



Dedication

This Guide is dedicated to Charles Bubb FIEAust 1928 – 2015. He was born and educated in Perth, WA. He graduated with a civil engineering degree from the University of Western Australia in 1950, and he joined the Commonwealth Department of Works and Housing. He worked with this Department until his retirement in 1987 finally as the Director of Engineering.

The 1968 M6.8 Meckering earthquake that destroyed the town of Meckering and caused damage as far away as Perth, changed the view on earthquakes of a number of structural engineers, and particularly for Charles Bubb. In 1969 with the support of the Department, he undertook earthquake engineering studies at Imperial College, London. On his return, he became a very active member of the Australian National Committee for Earthquake Engineering which was formed in 1971 and was responsible for the first national series of professional earthquake engineering seminars in 1974. The Committee also underpinned the development of Australia's first earthquake code AS2121 - 1979 which was led by Charles Bubb. He remained actively involved with earthquake engineering for most of his life, in his retirement playing a leading role in the establishment of the Australian Earthquake Engineering Society, of which he was the inaugural President, from 1990 to 1995.

Foreword

I am very pleased to introduce this new Guide to Seismic Design & Detailing of Reinforced Concrete Buildings in Australia with a focus on regions of lower seismicity such as Australia, on behalf of the Steel Reinforcement Institute of Australia. The Guide has been written for designers with a strong emphasis on the importance of systems thinking and practical detailing to improve the inherent robustness and resilience of reinforced concrete buildings. The Guide includes topics such as earthquake design philosophy, analysis methods and detailing of structural systems and components including connections, slabs, beams, walls and columns. It is imperative that designers ensure viable load paths exist in the structure from the roof level to the foundation with positive connections throughout. History has shown that earthquakes exploit the weakest link in structures as illustrated in this Guide with numerous case study examples.

The Guide is an ideal complement to the Australian Earthquake Loading Standard AS1170.4 and the Commentary published by the Australian Earthquake Engineering Society. The Guide strongly encourages designers to think how their structure will collapse under extreme lateral loading, and whether the collapse mechanism would lead to progressive collapse. Earthquake ground shaking causes buildings to displace laterally and buildings only collapse when the gravity load carrying elements can no longer sustain the drift imposed. The drift performance of buildings is improved through better conceptual design and detailing and also importantly in limiting the axial stress levels on the gravity load carrying elements. Designers can design for earthquake loading and demonstrate earthquake compliance using either the traditional force-based approach or a more contemporary displacement-based approach with an emphasis on drifts.

The Guide has been written by very experienced and eminent practising engineers with a very practical focus, and I highly commend the publication to you and wish you well in your future designs.

Prof John L Wilson PhD

Executive Dean, Faculty of Science, Engineering and Technology, Swinburne University of Technology, Chairman BD6/11 responsible for the preparation of AS 1170.4 on behalf of Standards Australia

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Preface

In 1995 — in response to the publication of the second Australian earthquake Code* AS 1170.4 *Minimum design loads on structures, Part 4: Earthquake loads*¹ — the SRIA published the *Seismic Detailing for Reinforced Concrete Buildings in Australia* (RCD17)². There have subsequently been two revisions to the Concrete Standard AS 3600 *Concrete structures*³, while a new edition of the Standard AS 1170.4 (retitled *Structural Design Actions, Part 4: Earthquake Actions in Australia*)⁴ was published in 2007.

The SRIA's new Guide is both an update and an expansion of the revised 2001 release of the 1995 document; it covers design as well as detailing of reinforced concrete buildings for seismic actions. It will assist graduate engineers, practicing engineers and other designers with limited seismic experience, and senior engineers seeking to refresh themselves of the current developments and practical aspects of reinforcement design and detailing for seismic actions in Australia.

Since the release of these documents in 1995, there have been significant advances in analysis software, and our understanding of earthquake design has improved through advances in research, combined with knowledge gained from the critical evaluation of the actual performance of buildings during earthquakes. AS 3600 provides Australian designers with the design rules for earthquake design to meet the typically lower seismicity of Australia. Most commercial buildings in Australia incorporate insitu reinforced concrete, designed and detailed in accordance with AS 3600. Complying with the Standard for regions of low seismicity (such as Australia) deems the structure to have adequate strength and ductility as a life safety measure.

The successful seismic performance of a concrete structure requires the structural designer to understand the fundamental importance of the inseparable link between design and detailing. Suitable detailing is crucial to ensure that the structure will respond under seismic loading in the manner for which it has been designed. Time and time again, earthquakes have shown that a well detailed structure can perform well even when subjected to seismic design actions considerably in excess of those for which it was designed.

For low values of the assumed structural ductility factor, $\mu \leq 2$, detailing is required only in accordance with the body of AS 3600; for higher values of ductility factor, $2 < \mu \leq 3$, detailing is required to Appendix C of the Standard. Levels of ductility $\mu > 3$ are outside the scope of the Standard; for which design and detailing to New Zealand Standards is suggested.

This comprehensive Guide (for new buildings) covers some matters that are not discussed in any Australian text. It does not, however, cover all design situations or requirements, but offers an assortment of basic seismic principles, design advice, and fundamentals to assist

designers. It also suggests further study of the principles and practice of seismic design and detailing.

The information in the Guide focuses on the key, functional and practical aspects of seismic design and detailing of reinforcement, with references to specialist information. Technology and the striving for reduced design and construction times can shift the focus away from the vital reinforcement detailing phase of the project. The overall aim is to foster cost-effective, simple design solutions by giving the designer the practical detailing information to determine the requirements for the overall structural performance under seismic loadings, efficiently.

Unfortunately, in earthquakes many buildings continue to perform poorly and in some cases badly, because of poor design, poor detailing, and poor construction practices. This outcome was confirmed again in the Canterbury earthquakes in Christchurch, NZ in 2011.

The late Professor Tom Paulay in 1997 said:

“

- In many countries the design process is synonymous with sophisticated dynamic analysis.
- The high degree of precision achieved with the use of computers is often held in inordinate reverence, 'at the expense of basic conceptual choice.'
- By choosing an unambiguous load path, establishing a strength hierarchy, and providing well-thought details one can tell the building how to behave. The weak links selected in the hierarchy must be detailed to provide a rationally conceived plastic mechanism. The regions between these plastic hinges must be sized to remain elastic. Engineering choices can be based on simple static, elastic models rather than sophisticated dynamic analyses.
- Detailing is very often considered a subordinate, depreciated drafting activity with apparent lack of intellectual appeal. The exact opposite should be true in our practices. The detailing of potential plastic regions of the structural system is partly an art ... it relies on the feel for, and understanding of, the natural disposition of internal forces. It often invites innovations. Rational detailing of high quality will compensate for the crudeness inherent in our ability to predict the magnitude of earthquake-induced displacements

* The BCA was introduced in 1996 as the Building Code and to avoid confusion the word Standard was adopted for any Standards Australia publications.

Overview

This Guide includes a brief overview of the history of seismicity in Australia, the role of the Building Code of Australia (BCA) and Australian Standards in designing buildings for seismic actions.

The need for sound engineering practice, the expectation of clients, relevant Standard requirements, responsibility for design and applicable Standards requirements and other aspects are also discussed.

This Guide examines the lessons learnt from previous seismic events in Australia as well as major seismic events such as the Canterbury earthquakes in New Zealand. It focuses on the analysis, the design methodology and, most importantly, the reinforcement detailing aspects for concrete building structures in regions of lower seismicity where design for seismic forces nevertheless is required.

Seismic design, including detailing, applies for all structures in Australia. Just as vertical loads are either permanent or imposed actions, lateral loads can be either wind or earthquake. Even if wind actions are larger than the earthquakes actions, they are separate and completely different design cases, which must be separately considered.

Wind actions are common and well understood; they are essentially resisted elastically, and drift is usually not a problem. Earthquake actions are uncommon and not well understood; they are resisted inelastically (and drift can be a problem) and they are fundamentally different to wind actions due to their cyclical nature.

This Guide shows how the requirements of the current Standards can be met through the use of predominantly simple 'seismic' details and good detailing practice. Further, it shows how an appreciation of structural performance under seismic action will enable the structure to be designed to withstand the anticipated earthquake actions.

Excellent overseas texts on the design of buildings for seismic actions include:

- Booth, *Earthquake Design Practices for Building*⁵.
- Paulay and Priestley, *Seismic Design of Reinforced Concrete and Masonry Buildings*⁶.
- Priestley, Calvi and Kowalsky, *Displacement-based Seismic Design of Structures*⁷.
- Booth, *Concrete Structures in Earthquake Regions: Design and Analysis*⁸.
- Moehle, *Seismic Design of Reinforced Concrete Buildings*⁹.

Details of other referenced material are provided in the References.

1 Background

1.1 The history of Australian earthquake Codes and Standards

Australia's first earthquake Code* AS 2121, *The design of earthquake-resistant buildings* (known as the SAA Earthquake Code) was published in 1979 as a result of the 1968 earthquake at Meckering in Western Australia. It was based on the 1977 edition of the SEAOC Code (Seismology Committee, Structural Engineers Association of California), *Recommended Lateral Force Requirements and Commentary* and the International Conference of Building Officials, California, USA *Uniform Building Code* 1976 edition, but adapted for Australian conditions. It was both a loading and material Code in working stress design with detailing.

AS 2121 was replaced in 1993 by a new Code: AS 1170.4 *Minimum design loads on structures, Part 4: Earthquake Loads*¹, with the detailing set out in AS 3600. A completely revised edition of the Standard AS 1170.4 retitled *Structural design actions, Part 4: Earthquake actions in Australia*⁴ was published in 2007.

Importantly, although not included in AS 1170.4 but set out in the foreword to the Commentary on the Standard¹⁰ is the statement that:

“the design of structures to the Standard does not necessarily prevent structural or non-structural damage in the event of an earthquake.”

The Standard simply provides the **minimum** criteria considered to be prudent for the protection of life by minimising the probability of collapse of the structure or parts of it. This concept is discussed later in this Guide.

1.2 Seismicity in Australia

Australia has a history of moderate earthquakes by world standards, and although the risk is low, intraplate earthquakes can and do occur anywhere in Australia on a regular and recurring basis. There is no mathematical or geological model to determine where or when the next earthquake will happen.

Australia is located wholly within a tectonic plate and is subject to intraplate action. Intraplate earthquakes, although infrequent, can be larger than the moderately sized interplate earthquakes that regularly occur along the plate boundaries in locations such as California and New Zealand.

In Australia, significant intraplate earthquakes have been recorded since 1788 though often occurring in relatively unpopulated areas. The 1954 earthquake in Adelaide (magnitude 5.4) was a moderate earthquake that occurred in a location south of the city in a low population area, but still caused extensive damage. The 1988 earthquake in Tennant Creek (magnitude 6.3 to 6.7) was large by world standards and would have caused significant damage had it occurred in a major city. Possibly Australia's largest earthquake series occurred in 1889 off the north-east coast of Tasmania (estimated magnitudes of 6.3, 6.6 and 6.9).

The 1989 Newcastle earthquake (magnitude of 5.6) was a relatively small intraplate event but caused significant damage and loss of life. However, this was not the first earthquake in Newcastle; several had occurred previously.

The history of earthquakes in Australia has been modest. In the past 80 years, there have been 20 earthquakes of magnitude MM6 or greater on the Modified Mercalli Scale[#] in Australia. Today, seismologists suggest that a major event of M7[^] or greater might occur every 10,000 years in Australia and that it is only a matter of time before the next significant earthquake occurs. The Newcastle earthquake is the only earthquake resulting in loss of life in Australia to date. For a list of major earthquakes in Australia, refer to <http://www.ga.gov.au/scientific-topics/hazards/earthquake/basics/historic>.

Australia is one of the more active intraplate areas in the world. All seven large earthquakes in Australia since 1968 have surface faulting with fault scarps of up to 37 km in length and vertical displacements of up to 2 m.

Seismicity also varies across Australia because of the different geological eras of the formation of the continent. Several hundred seismic events occur in Australia alone each year, but most are magnitude M2.5 or lower.

Although Australian earthquakes are often of low Moment Magnitude scale, M (ie the energy released at the epicentre of the earthquake), they are typically shallow in depth, at between 5 km and 20 km below the earth's surface and of relatively short duration. The energy received at the surface can, therefore, be reasonably large but is over a smaller area. Typically, interplate earthquakes such as in New Zealand can occur between 20 km and 100 km below the earth's surface. The severe earthquake in Christchurch (called an after-shock) in 2011 was only 5 km deep and, as a result, the energy released at the surface was enormous, resulting in extensive damage and loss of life.

* The BCA was introduced in 1996 as the Building Code and to avoid confusion the word Standard was adopted for any Standards Australia publications.

Modified Mercali Scale (denoted as MM) measures the effect of the earthquake at the surface of the ground. It is not as scientific as the Moment Magnitude scale, which measures the energy at the epicentre of the earthquake.

[^] The Richter scale has been replaced by the Moment Magnitude Scale (abbreviated as MMS; denoted as MW or M). It is used by seismologists to measure the size of earthquakes in terms of the energy released at the epicentre of the earthquake. This scale was developed in the 1970s to succeed the local magnitude approach of the 1930s-era or better known as the Richter magnitude scale (abbreviated as ML).

On average Australia will experience:

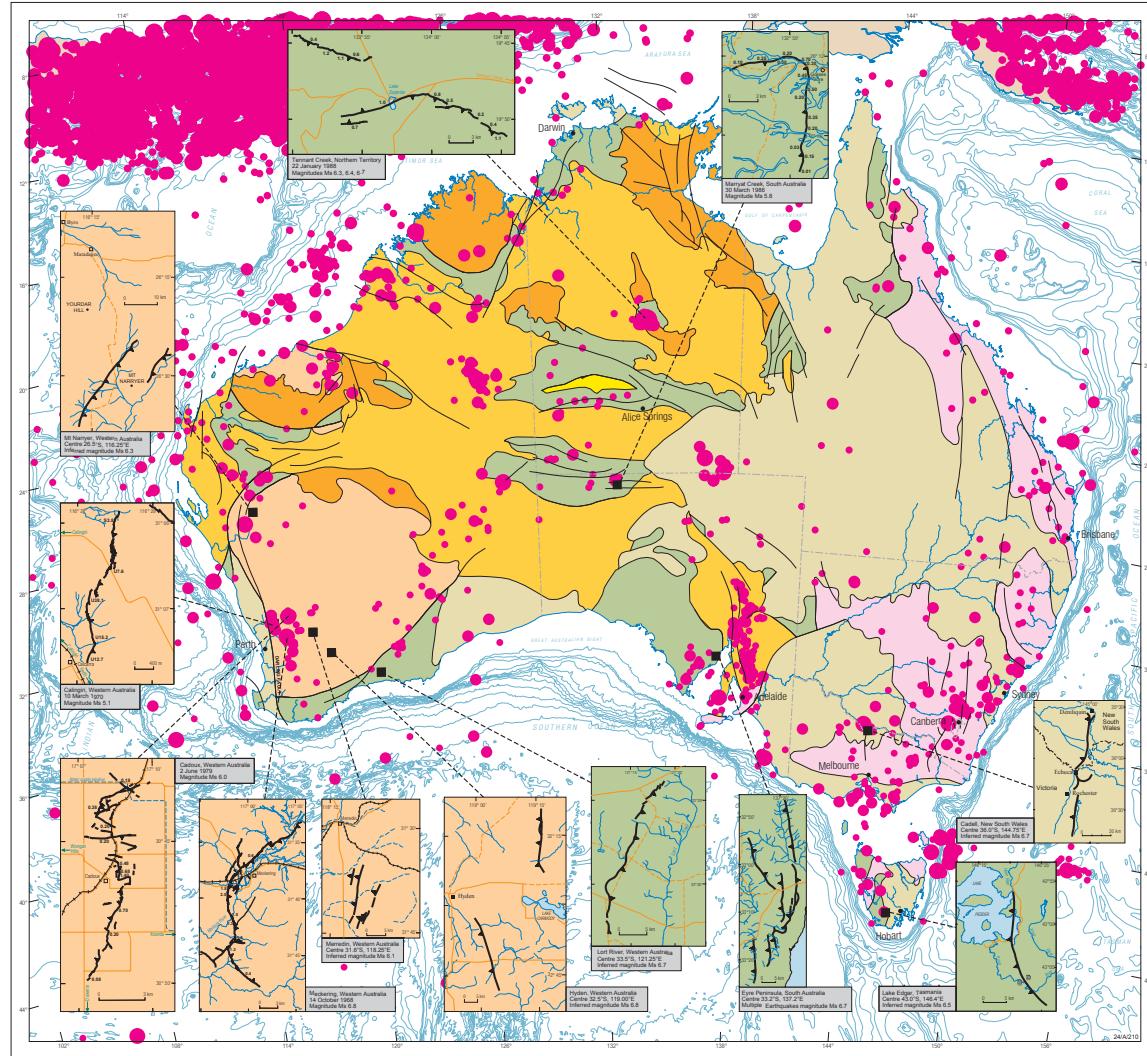
- 1 shallow earthquake of magnitude 6 or more once every 10 years (equivalent to the 2011 Christchurch earthquake)
- 1 shallow earthquake of magnitude 5 or more every 2 years (equivalent to those in Newcastle and Adelaide)

Unfortunately, there continues to be a perception amongst building owners and developers, builders, contractors, subcontractors, engineers, architects and designers that earthquakes do not occur in Australia. The reality is they are a regular occurrence, but the return periods can be long, and often occur in isolated areas.

Significant earthquakes have been recorded in all States and Territories Figure 1.

To date, Australia has been lucky with its earthquakes as two of the devastating ones at Meckering and Newcastle have both occurred on public holidays when many buildings were unoccupied. Had the 1989 Newcastle earthquake occurred during a typical working day, the number of deaths would have been 200 to 300 or more, not the 13 recorded.

The Newcastle earthquake of moment magnitude scale M5.6 was a moderate earthquake by Australian experience. It was, however, one of Australia's most expensive natural disasters, costing about \$5bn in



This map of active faults was compiled as part of a national program to map active faults in Australia. The Australian continent is completely within the Australian Plate and consequently there are no major through-going active faults like the Alpine Fault in New Zealand, the Mediania Fault in the Philippines or the San Andreas Fault in California. However, there are many smaller active faults such as the Darling Fault along the south-west edge of the Yilgarn Block in Western Australia and the Great Escarpment in South Australia. There have been many small earthquakes along it in the last 15 years. In 1957 BMR (Geoscience Australia) installed a network of seismic stations across the Great Escarpment to monitor any movement. The same goes for all the faults shown on the map.

Since 1978, seven large shallow earthquakes (including three at Territorial Creek) have ruptured the Great Escarpment. Central Australia has very few unruptured faults causing more than half of the world's known intraplate fault scars. Another seven fault scarps have been identified in the last 15 years. Most of these are in the West Eastern Australia during routine geological mapping but many of these have not as yet been investigated or mapped in detail and only the location and inferred magnitudes of the associated earthquakes are known.

Cronin & others (1992) trench the Territorial Creek and Murray Creek fault scarps and concluded that there had been no earlier movement on these faults. They also found no evidence of earlier movement on the Collier Bay and Tasmania has demonstrated that it is younger than 10 000 years old, and ruptured by at least two earthquakes in that time (McCuie and others, 1996).

Cronin & others (1992) and Macchette & others (1992) trench the Territorial Creek and Murray Creek fault scarps and concluded that there had been no earlier movement on these faults. They also found no evidence of earlier movement on the Collier Bay and Tasmania has demonstrated that it is younger than 10 000 years old, and ruptured by at least two earthquakes in that time (McCuie and others, 1996).

Macchette & others (2000) can predict where or when the next large earthquake will occur. There were magnitude 6 or greater earthquakes in Australia in the 20th century. The last at Collier Bay, Western Australia on 10 August 1997.

@Seismicisolation



Figure 2 – The Newcastle Workers Club, NSW

(Photograph courtesy Newcastle Council Earthquake Collection, Newcastle Region Library Ref No. 046 000 544)

current value. There were 13 deaths, more than 160 people were injured, and about 40,000 buildings were damaged - most of them being older building stock.

At the time of the Newcastle disaster, earthquake design of buildings was generally not required in Australia^{11 12}. The Newcastle Workers Club, Figure 2 which was designed in the early 1970s collapsed during the earthquake. It frequently held major events attended by hundreds of people. Failure was attributed to the progressive collapse of the concrete columns on the western side of the building due to errors in design, incorrect detailing of the reinforcement, and lack of understanding of the effect of earthquakes on such buildings^{13 14}. The building was subsequently demolished and rebuilt. Much of Australia's existing building stock would perform in a similar fashion to that in Newcastle. Many older buildings especially unreinforced masonry buildings (URM) are the most vulnerable to seismic action.

As a result, Australia is considered a 'low-risk' but 'high-consequence' country in terms of earthquake damage, ie, the probability of an earthquake occurring in a major city is low, but the consequences, should one occur, are likely to be dramatic. Furthermore, it must be remembered that, on average, Australia experiences earthquakes of moment magnitude 6 or greater every 10 years. Such events have ground motion amplitudes 2.5 times and released energy at least 4.0 times as great as the Newcastle earthquake Figure 2.

1.3 Reinforced concrete structures and earthquakes

The Canterbury earthquakes in New Zealand in 2010 and 2011, the Kobe (Great Hanshin) earthquake in Japan on 17 January 1995 and the Northridge earthquake in Los Angeles on 17 January 1994 were significant and large earthquakes. The study of how buildings perform during these events has highlighted the strengths and weaknesses of reinforced concrete in terms of design and as a structural material Figures 3 and 4.



Figure 3 – Damage caused by the Kobe earthquake, Japan

(Photograph courtesy John Woodside)

Reinforced concrete has a number of attributes that allow it to be successfully employed in structures resisting seismic actions, viz:

- Properly conceived and detailed concrete structures have excellent ductility in flexure, and it can equal that of structural steel.
- Well-confined concrete can have good ductility under flexure and axial compression, with a lower tendency for buckling failure compared with an equivalent steel structure, as seen in a car park in the 1994 Northridge earthquake (magnitude M6.7)
Figure 4.
- Appropriately detailed concrete construction provides a monolithic structure, which contributes to good overall continuity and load path redundancy, in itself a good earthquake-resistant feature.
- Shear (structural) walls are an economical means of providing high lateral strength and stiffness while retaining significant ductility. They also have the advantage that they limit drift and reduce damage to non-structural parts and components. Many buildings will have reinforced concrete lift shafts and stair shafts which will act as shear walls.
- Internal damping before yielding is greater than in steel structures. Damping is important for serviceability considerations during moderate earthquakes.
- In many countries, including Australia, reinforced concrete is the building material of choice; the technology is familiar, the materials are locally available and cost-effective, while the finished structure will have good acoustic and thermal properties.

Designers, however, should recognise that in terms of seismic behaviour, reinforced concrete has a few drawbacks when compared to steel or timber structures. These include:

- A high mass-to-strength ratio;
- Brittle behaviour when allowed to fail in shear – particularly for low levels of shear reinforcement;
- Lack of ductility in compression when the reinforcement is inadequately confined.

The mass-to-strength ratio is important because earthquake design actions arise from inertial effects and are proportional to the mass, so concrete is at a weight disadvantage compared to steel and timber. The brittle behaviour exhibited in shear can be overcome by providing a sufficient reserve of strength to suppress such failures, while providing adequate transverse confining reinforcement steel significantly increases the ductility of concrete in compression.

The commonly held view that modern steel structures are immune from collapse or significant damage during seismic events is not correct. In Christchurch, a number of steel structures suffered extensive damage or collapse and in the Northridge earthquake in the USA, many welded connections failed.

Many well-designed concrete structures have survived major earthquakes in Canterbury, Kobe, and Northridge undamaged. Readily available literature provides a wealth of experimental and theoretical evidence to support the good seismic performance from reinforced concrete. Good detailing of reinforced concrete structures will enhance the '*strengths*' of the structural system while minimising the potential weaknesses identified above.



Figure 4 – Ductility of concrete columns in the Northridge earthquake, California USA.

(Photograph courtesy EERC/University of California.)

1.4 The BCA and Standards

The Building Code of Australia¹⁵ (BCA) forms Volumes 1 and 2 of the National Construction Code (NCC) and specifies minimum mandatory performance requirements. It also contains the provisions for the design and construction of buildings and other structures in Australia and provides deemed-to-comply solutions. Alternative solutions allow for innovative design and use of materials; such solutions require a combination of documentary evidence, verification comparative analysis and certification by an expert in a particular field.

Usually, the deemed-to-satisfy solutions from the BCA are used. It is usually simpler to follow this path and use the referenced Standards.

The BCA references the AS 1170 series of Standards and AS 3600 to prescribe the minimum requirements for the design of concrete structures. AS 3600 also references other Standards.

Actions (loads) on buildings are determined and applied in accordance with AS/NZS 1170, a series of five Standards covering the whole range of loading conditions in Australia and New Zealand.

- AS/NZS 1170.0 covers the general principles and sets out the various actions (loads) and combinations of actions (load combinations) in the design for ultimate and serviceability limit states.
- AS/NZS 1170.1 covers permanent (dead loads) and imposed actions (live loads).
- AS/NZS 1170.2 covers wind actions.
- AS/NZS 1170.3 covers snow actions.
- AS 1170.4 and NZS 1170.5 respectively apply to Australia and New Zealand and cover earthquake actions.

The referenced Concrete Standard is AS 3600. Other Standards not called up directly by the BCA but referenced in a Standard called up by the BCA, must also be complied with for design. Standards not referenced or called up in the BCA are like a handbook or manual; their use is optional. An example would be AS 3826¹⁶ on the seismic upgrading of existing buildings.

1.5 Risk mitigation and minimising damage to buildings

Ziggy Lubkowski, Arup, stated that:



How many engineers tell their clients that a code-based design could leave their investment damaged beyond repair? Current building codes do not focus on earthquake resilience – the ability of an organization or community to quickly recover after a future large earthquake. The code's objective is only to protect the lives of building occupants.

This means significant damage to the building structure, architectural components, facades and other elements is allowed as long as the code objective is met. It is therefore not surprising that when a major earthquake strikes an urban region the losses are large and the general public is left to wonder why. The Christchurch earthquake in February 2011 is a prime example of this.

The traditional focus of earthquake design worldwide is to preserve life, not minimise damage to building. This philosophy is used in AS 1170.4 by assuming an inelastic structural response under the design earthquake action. This allows the designers to use reduced design actions for earthquakes by an order of 30% to 60% than might be expected in a large earthquake. A properly designed and detailed building **for life safety** will enable occupants to evacuate the building. However, it is likely to result in damage to the building, requiring either repair or, in an extreme event, demolition.

Unfortunately, society's expectation may not match the minimum requirements of the Standards. This has been the response expressed by building owners and the public in Christchurch. For instance, does a building in the event of an earthquake require protection of irreplaceable contents, eg a museum? Is there a need for the continuing use of the building after the event? eg a hospital, or in the extreme condition does it contain dangerous materials, eg a biological laboratory dealing with dangerous viruses? These are issues that the designer must consider early in the design phase.

The lowest level of protection is the BCA requirements where the building is expected not to collapse and can be evacuated in the event of a significant earthquake. This approach, however, does not prevent damage or possible demolition in an extreme event.

Alternative methods are being proposed to reduce the risk of damage sustained in earthquakes. These methods include some new technologies to minimise damage¹⁷.

The highest level of protection for a building currently available is base isolation. This technique can reduce the risk of damage to the building structure and its contents and can provide a fully operational building in a major event. This approach has been available for many years and used successfully overseas. The order of increased construction cost is thought to be of the order of 8% to 10%.

The next level of protection is to try to minimise the damage by using a more robust and regular structure with a higher level of ductility. This level should minimise the damage to the primary structure even in the most severe earthquake with many alternative load paths and backup systems with the ability to resist greater forces than the minimum required by the Standard. The great advantage of this approach is that the structure should remain operational and be repairable. Insurance premiums may be less, and the mitigation of the risk to structural damage and business continuity is achieved but at an increased construction

cost of the concrete structure. It is thought that the increase would be as little as 1% to 3% over that of providing the lowest level of protection required by the BCA. This assumption is based on the structural cost being about 25% of the total cost of the building, and the additional design and detailing would result in an increased structural cost of the order of 5-10%.

Safety in design may also warrant a higher level of protection from the minimum design level, for example for a nursing home or regional hospital.

1.6 Sound engineering practice

John Carpenter, former secretary SCOS* stated that:

“Good design is risk led, rather than code dependent.”

Bill Boyce, former Associate Professor Civil Engineering, the University of Queensland stated that:

“We seem to be too driven, and limited, by Standards and Codes (and I am not denying their usefulness) so that deep understanding of structural behaviour is not seen as vital – provided we satisfy the rules in the Standards we believe we have done all that is necessary.”

Standards represent the technical information current at the time of publication but can become out of date. A better understanding of the design issues, the performance of structures in recent earthquakes and new research and testing has increased the designer's knowledge of the performance of buildings under earthquake actions.

Standards often do not include all design matters that need to be considered in detail such as floor diaphragms discussed in Section 5.10.

AS 3600 sets out only the **minimum** requirements for the design and construction of concrete structures. These requirements have been developed over many years and draw on the experience embodied in international standards and on local expertise. In general, the rules have been established to protect the public (safety) and provide a general level of amenity. However, such rules are not sufficient to prevent damage or structural failure to the structure of a building during a significant earthquake.

Sound engineering practice is where the principles of engineering that are broadly accepted by engineers at the time of design, and which may be additional to minimum legal requirements, are adopted.

Sound engineering practice involves identifying the load paths of compression and tension forces through beam-column and beam-wall joint zones and at junctions between other structural elements under cyclic loading conditions and detailing them appropriately.

Clearly in complying with sound engineering practice, designers must comply with the fundamental assumption on which all structural design is based, ie that every action or inertial force must have an adequate load path or paths from its point of application to the foundations, in which equilibrium of forces and compatibility of strains are satisfied.

1.7 Ductility

Ductility is a vital characteristic of the seismic resistance of buildings. Ductile materials and connections will deform but not fail even when subjected to design actions beyond those required by Standards. This behaviour is fundamental to designing robust buildings for seismic actions.

The definition of ductility (of a structure) used by AS 1170.4 is the numerical assessment of the ability of a structure to sustain its load carrying capacity (anticipated energy) when responding to cyclic displacements in the inelastic range during the earthquake. *The structural ductility factor, μ , is the ratio of the overall deflection of the structure prior to failure to the deflection at yield, $\mu = \Delta_u/\Delta_{yu}$* Figure 5. The ductility factor, μ , depends on the structural form, material ductility and damping.

The structural ductility factor, μ , is specified in Table 6.5 (A) of AS 1170.4 and Table C3 of AS 3600.

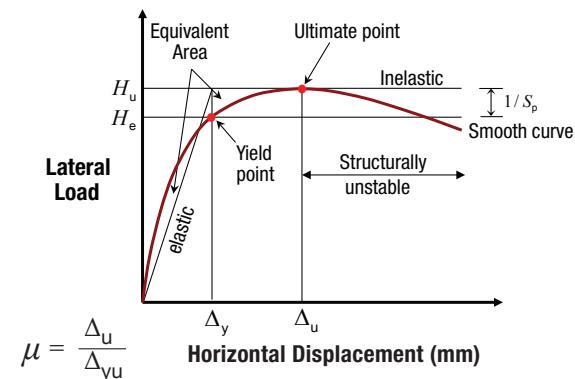


Figure 5 – Structural Ductility Factor, μ .

The structural performance factor S_p is a numerical assessment of the additional ability of the total building (structures and parts) to survive earthquake actions.

The factor $1/S_p$ represents the available over strength in the structure, and the factor μ/S_p is equivalent to the structural response factor (R_f in the previous version of the Standard). The higher the factor μ/S_p , the greater the inelastic response assumed by the designers, which has the effect of reducing the design earthquake actions, but comes at the cost of requiring greater attention to detailing to ensure the associated increase in ductility demands can be achieved.

* CROSS (Confidential reporting on structural safety) and SCOS (Standing committee on structural safety) is sponsored by the Institution of Structural Engineers, the Institution of Civil Engineers and the Health and Safety Executive UK.

1.8 The client/building owner's expectations

The needs of a particular owner for a given structure may require more stringent requirements than those set out in AS 1170.4 and AS 3600. Early in the preliminary design phase, designers should ascertain the owner's expectations of the building in the event of a major earthquake. Should designers consider whether a life safety or low-damage design is appropriate. Also, whether or not, a cost-benefit analysis of a more robust design is appropriate.

Commonly, clients/building owners have a different view on what earthquake design means for their building. They mistakenly assume that their building will survive a major earthquake without damage, which is not the intent of the Standards.

Clients/building owners should be advised that while the probability of earthquakes is low, the damage can be extreme compared to high winds where the probability is high but the damage is usually moderate. Many businesses failed after the Newcastle earthquake because they were unable to continue to trade; income ceased, but costs did not. Those without business interruption insurance often failed, and they were sometimes unable to access their premises or recover valuable data.

The greatest reinsurance risk in Australia is for a large earthquake event in a major city such as Sydney or Melbourne with significant damage. While this is an unlikely event, the damage and loss of life will be very extensive, should a large earthquake occur, and the costs will be high.

The designers of the structural system should also consider offering design solutions taking advantage of the ductility and energy dissipation of reinforced concrete, so that damage is allowed to occur at predetermined locations where the damaged components can be easily replaced at minimal cost and time so that the time the building may be out of action is minimised.

The design of non-structural parts and components such as building services, and architectural elements such as walls, windows, and ceilings are discussed in **Section 8**. Non-structural parts and components are equally important as the structure and designers of these items must also be involved in this discussion. The failure of non-structural elements and components cause harm to occupants and the public outside, prevent evacuation and often result in an inoperative building.

1.9 Australian Standard's seismic requirements

For a domestic structure having a height more than 8.5m to the top of the roof and for a non-residential reinforced concrete building, the seismic design actions are determined using AS 1170.4 and detailed in accordance with the Concrete Standard AS 3600.

The requirements of AS 1170.4 allow for the possible seismic forces that a concrete building structure in Australia may reasonably be expected to undergo at some point in its life.

The seismic design involves the following steps:

- Determine the Importance Level for the structure (AS/NZS 1170.0 Structural design actions, Part 0: General principles, and the BCA).
- Determine the annual probability of exceedance as set out in the BCA.
- Determine the probability factor, k_p , and the hazard factor, Z , (Section 3 of AS 1170.4).
- Determine the site sub-soil class (Section 4 of AS 1170.4). A geotechnical engineer will usually provide this advice.
- Determine the Earthquake Design Category (EDC) from Table 2.1 of AS 1170.4. (There are categories I, II and III.)
- Design the concrete structure in accordance with the requirements for the chosen EDC as set out in Sections 5, 6 and 7 of AS 1170.4. Note that there are height limitations of 25 m and 50 m, and those structures that above those height limits requires a higher EDC.
- Design and detail the reinforced concrete structure in accordance with AS 3600 and determine if detailing is in accordance with the body of the Standard or Appendix C. This will be related to the level of ductility assumed in the analysis.

The Importance Levels for buildings are set out in AS/NZS 1170.0 and the BCA, which provides examples of the various building types for each Importance Level. Buildings of Importance Level 1 (IL1) are not required to be designed for earthquake actions but still should have a reasonable level of ductility and robustness.

Buildings or structures classified as Importance Level 2 (IL2) are buildings and facilities such as low-rise residential construction or buildings and facilities that are below the limits for Importance Level 3 (IL3).

Larger commercial and institutional buildings will have an Importance Level of IL3 or IL4. Importance Level 4 (IL4) is reserved for buildings or structures that are essential facilities or with a special post-disaster requirement, eg a major hospital.

AS 1170.4 allows a higher level of analysis to be used than that specified in Table 2.1 of AS 1170.4 for a particular Earthquake Design Category (EDC) and this is commonly used, as the software is readily available for such analysis.

All structures have to be designed for both wind and earthquake actions. Although wind actions may exceed calculated earthquake actions, the results of the earthquake may be more critical. If wind actions are larger than earthquake actions, the earthquake actions and detailing must still be considered because:

- It is a requirement of the BCA to design for both actions as they are separate load cases and are completely different;
- The significant level of crudeness by which earthquake actions are determined;
- Columns and walls are the most important structural elements in buildings and failure of these will usually lead to collapse.

Because AS 3600 is based largely on ACI 318-14¹⁸, reference is made to the American Standard ACI 318 in a number of places in this Guide, where clarification or further information is provided for designing and detailing for earthquake actions.

Appendix C is an integral part of AS 3600 and designers must refer to this Appendix to determine whether they can design and detail the structure in accordance with the main body of the Standard or whether they have to use the additional requirements of the Appendix C. Designers have to make a fundamental decision as to the structural system they adopt and the structural

ductility factor μ selected. The various structural systems will typically involve moment-resisting space frames and/or shear walls.

Moment-resisting frame systems (MRFs) are structural systems in which an essentially complete frame supports the vertical actions and the horizontal earthquake forces by both flexural and axial actions of members and connections. These systems assume no shear walls.

Three types of MRFs are defined in the Standard, viz Ordinary Moment-Resisting Frames (OMRF); Intermediate Moment-Resisting Frames (IMRF) or Special Moment-Resisting Frames (SMRF).

Shear wall systems are systems in which either loadbearing or non-loadbearing shear walls provide the horizontal earthquake resistance acting in the plane of the wall. The concrete walls to the lifts and stairs are predominantly used as shear walls in many Australian buildings. The use of shear wall systems allows simpler analysis and does not involve the additional detailing associated with moment-resisting frames, refer **Section 5.6**. However, moment-resisting frames can be included in the structural system, as a combined system.

The structural ductility factor, μ , and the structural performance factor S_p for the various structural systems, are set out in Table 6.5 (A) of AS 1170.4 and Table C3 of AS 3600. This data is combined and shown in **Table 1**.

Once selected, the structural system must be designed and detailed to ensure that the system will behave in the way intended. Where the structure is of mixed construction, the lowest value should be adopted to reflect the ductility available from the less robust components.

Table 1 – Structural Performance Factor (S_p) - Basic Reinforced Concrete Structures
(After AS 1170.4 and AS 3600)

Description	μ	S_p	S_p/μ	μ/S_p
Special moment-resisting frames (fully ductile)*	4	0.67	0.17	6
Intermediate moment-resisting frames (moderately ductile)	3	0.67	0.22	4.5
Combined systems of intermediate moment-resisting frames and ductile shear walls designed in accordance with AS 3600	3	0.67	0.22	4.5
Ordinary moment-resisting frames	2	0.77	0.38	2.6
Ordinary moment-resisting frames in combination with limited ductile shear walls	2	0.77	0.38	2.6
Ductile coupled walls (fully ductile)*	4	0.67	0.17	6
Ductile partially coupled walls*	4	0.67	0.17	6
Ductile shear walls	3	0.67	0.22	4.5
Limited ductile shear walls designed in accordance with the body of AS 3600	2	0.77	0.38	2.6
Other concrete structures not listed above	1.5	0.77	0.51	1.95

* The design of structures with $\mu > 3$ is outside the scope of AS 3600, designers are referred to NZS 1170.5 and NZS 3101.

1.10 Obligations of designers and supervisors

The division of responsibility between the parties involved in the design and those in the construction of a building must be clearly understood. Most building projects will involve a number of design disciplines such as architectural, structural, civil, mechanical and electrical.

The supervision of construction is the responsibility of the builder or contractor. Preferably, the designers of the building should inspect the construction of the works on a periodic (regular) basis to ensure that the intent of their seismic design is being met. Inspections by the builder or contractor using QA procedures are no substitute for periodic inspections by the designer. Builders and contractors are not designers and nor do they have the necessary knowledge, understanding or experience in the design and detailing of buildings for seismic actions.

A further obligation that should be considered is that the drawings for the building must be kept by the designer and/or the statutory authority so that in any post-disaster search-and-rescue operation they will be accessible and provide to those involved, an understanding of how the building was designed and constructed.

1.11 The responsibility for earthquake design

It is recommended that for the design and detailing of a concrete structure for seismic actions, one structural engineer (called the *Principal Designer* in this Guide) must take overall responsibility for the structural aspects of the project, even though there may be a number of designers working on the project (this concept also applies to other disciplines). This unity of design responsibility will ensure that the building integrity is achieved by providing continuity of the overall structural system and integration of individual elements to act as a complete unit under seismic actions. Graduate or inexperienced structural engineers, while designing concrete under direction should not have overall responsibility for the design or detailing of a concrete structure for earthquake actions.

The *Principal Designer* responsible for the structure must be suitably qualified and a practising civil or structural engineer, eligible for Chartered Status of Engineers Australia or equivalent and experienced in the design and detailing of concrete structures for earthquake actions of comparable importance.

For larger structures, the approach for earthquake design and detailing should be peer-reviewed independently early in the project by someone with experience in structural and earthquake design to confirm that the

design approach is appropriate and that the extent of detailing to be used is adequate.

The collapse of the CTV building in Christchurch **Figure 6** where 115 people lost their lives has been attributed to the designer not being sufficiently experienced in earthquake design and not fully understanding what was required. Furthermore, his work was not properly overseen by his senior engineer*.

The study of selected buildings in Christchurch, some of which failed in the 2011 Canterbury earthquake¹⁹, exposed design mistakes and the poor detailing of the reinforcement used leading to their failures. Correct design and detailing of reinforcement was once again shown to be fundamental to satisfactory earthquake performance.

The *Principal Designer* and the design team should preferably carry out all the structural design of the building. Where part of the design is assigned or subcontracted to others, the *Principal Designer* needs to understand and fully coordinate those designs and take overall responsibility for them. The *Principal Designer* should ensure that this work is correctly and adequately specified and coordinated by way of detailed performance requirements that include integration of the manufacturer's components with the structure as a whole. Examples of design by others requiring detailed integration and coordination are the design of precast concrete elements and post-tensioned floors. This integration and coordination includes:

- Ensuring that the structural requirements are fully coordinated with the overall design;
- Confirming that sufficient preliminary design has been carried out so that structural sizes established are appropriate for the final design;
- Setting out the design requirements and design criteria including the design actions, earthquake requirements, fire ratings, covers, and the like;
- Confirming that any earthquake detailing is properly coordinated as the external designers or the subcontractors cannot be expected to understand the overall design of the building or how earthquake actions are to be resisted. This is particularly critical at interfaces and connections, including tie bars and collectors in slabs to connect the columns, and walls to the diaphragms.
- Care needs to be taken to ensure that non-loadbearing precast elements do not interact with the primary lateral load-resisting system adversely affecting its behaviour under seismic actions.

Functional requirements such as the location of columns and walls, stairs, lift shafts, building services and penetrations through the structure and their effect on the earthquake design must be considered early in the design process.

* In the failure of the CTV building in Christchurch in New Zealand where 115 people lost their lives, the Royal Commission concluded that there were a number of non-compliant aspects of the CTV building design. They concluded that a primary reason for this was that the design engineer was working beyond his competence in designing this building. He should have recognised this himself, given that the requirements of the design took him well beyond his previous experience. They also considered that his superior was aware of the engineer's lack of relevant experience and should therefore, have realised that this design was pushing him beyond the limits of his competence.

If changes are made to the drawings during construction, the *Principal Designer* or an equivalent senior designer must be responsible for reviewing and approving such changes.

In a similar manner, if there are changes to the structure of a building, after it has been constructed, any design changes and new construction must recognise the original and current earthquake requirements and comply with current design requirements.

1.12 Site sub-soil class

There were significant changes to the BCA in 2010 with the adoption of the 2007 version of the Earthquake Code AS 1170 .4, including the adoption of five site sub-soil conditions and different conditions for many cities, notably in Adelaide where soil amplification effects of the deep stiff clay profile was first recognised.

Soft soils such as sands and silts accentuate the effects of earthquakes. Liquefaction was observed in early Australian earthquakes, particularly the Beachport earthquake in 1897, but to date, there have been no reports of foundation failures in Australia, unlike in the Canterbury Earthquakes. AS 1170.4 does not consider the effects of liquefaction.

1.13 Canterbury Earthquakes, New Zealand

The final report of the Royal Commission of Inquiry²⁰ wrote the following words on the Canterbury earthquakes:

On 4 September 2010, at 4:35 am, an earthquake of magnitude 7.1 struck Christchurch and the surrounding Canterbury region. The earthquake had an epicentre near Darfield, a small town about 40 km west of the Christchurch Central Business District. An aftershock sequence began, which at the time of writing is ongoing. All of the earthquakes were the result of ruptures on faults not known to be active prior to the September event.

The early morning timing of the September earthquake and the rural location of its epicentre no doubt prevented fatalities. However, many unreinforced masonry buildings were damaged and there was extensive damage to infrastructure. The eastern suburbs of Christchurch and Kaiapoi were seriously affected by liquefaction and lateral spreading of the ground.

The September earthquake was followed by four other major earthquakes occurring on Boxing Day 2010, and 22 February, 13 June and 23 December 2011. Of these, the event on 22 February was by far the most serious, resulting in 185 deaths. It led to the establishment on 11 April 2011 of this Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes. **Figure 6.**



Figure 6 – The ruins of the CTV Building, Christchurch, New Zealand

(Photograph courtesy Peter McBean)

While the extent of damage due to the series of earthquakes in Christchurch in 2010 and 2011 is unlikely to occur in Australia, the reports by the Royal Commission are sobering reading, as 185 people lost their lives in the 22 February 2011 earthquake. Similar or greater loss of life could be expected in Australia should a major earthquake occur in one of the major cities.

The University of Canterbury prepared a report on the general building performance in the Christchurch CBD¹⁹. This report summarises many fundamental issues and recommendations, some of which are relevant in Australia, albeit on a lower scale of risk. Failures of buildings have highlighted a number of issues including:

- Poor detailing of reinforced concrete.
- Errors in design.
- Failure to recognise that 'irregular' buildings will put much more additional earthquake actions into a structure than has been previously understood. In particular, vertical irregularity caused problems, predominantly in shear wall structures.
- The failure of diaphragms due to poor detailing and their inability to transfer forces into the vertical loadbearing elements.
- The failure of stairs and the need to provide seismic joints so that the stairs could move under earthquake actions in all directions and for stairs to be designed for the drift.
- Problems with concrete buildings including precast flooring systems in conjunction with ductile frames. Problems include the precast ducts used to grout the starter bars from the panels below often not being laterally restrained or within an area of confined concrete, lateral restraint of reinforcement in walls is often inadequate, and the use of T and L shaped wall systems which performed poorly.
- Inadequate seismic gaps in precast panels and failure to ensure that these gaps did not become filled with solid sealants, mortars and the like.
- Non-structural components performed badly due to inadequate attention to detailing and design.

Professor Des Bull spoke on some of the lessons learnt from the Canterbury earthquake series at an Asian Pacific Forum on *Structural Engineering for Extreme Events* in 2011²¹ with similar comments. These lessons included:

- Reinforced concrete walls with two layers of reinforcement and boundary elements performed much better than walls with a single layer of reinforcement in the middle of the wall.
- The failure of concrete transfer structures supporting walls and columns above.
- Failures of laps in reinforcement where the concrete was extensively damaged and spalled.
- The failure of diaphragms and the issues with beam elongation leading to loss of support of the floors.
- The poor performance of stairs and ramps.

2 Case Studies

Two concrete buildings affected by the Canterbury earthquakes in New Zealand are reviewed to illustrate good and not so good design practices²² together with a concrete building in Australia representing sound engineering practice.

2.1 Central Police Station, Christchurch

The Christchurch Central Police Station was designed in 1968 to very old codes of the day. As a Government building, it was designed as an essential post-disaster building and constructed to more rigorous design requirements than the minimum specified at that time.

The 15-storey reinforced concrete building had three levels of podium which was about twice the plan area of the tower above and it was regular in construction both in plan and elevation Figure 7.

The vertical and lateral forces were resisted by insitu reinforced concrete ductile moment-resisting frames. Precast concrete panels were used as non-structural cladding elements and also for some other walls, and there were no shear walls. The precast walls had seismic gaps to that they were not part of the structure resisting the seismic actions. The stairs were fixed to the floors at their upper level but allowed to slide at their lower level.

Detailing of the reinforced concrete structure was of a high standard even though it was designed in 1968. While not meeting current standards, in many cases it was similar. From structural details, the columns were designed to be considerably stronger than the beams so that the concept of strong column weak beam was achieved. The detailing of the beams and columns was such that plastic hinges, should they form, occurred in the beams rather than the columns.

Although the building suffered some damage, mainly non-structural, it was still usable after the earthquake showing that adequately reinforced concrete buildings can perform satisfactorily even under major events. While the building was assessed as being structurally sound following the earthquakes, it was vacated some months later over concerns that aftershocks may affect the building's internal services. As it was considered uneconomic to repair the building, asbestos was removed and it was demolished in 2015.



Figure 7 – Central Police Station, Christchurch, NZ
(Photograph courtesy Eric Lume)

The Canterbury Earthquake Royal Commission concluded for this building:

- //
- *The performance of the building in the earthquakes was very satisfactory in terms of the structural damage that occurred. The very robust nature of the building, due to its high level of redundancy and its symmetrical and regular form is noteworthy.*
 - *The detailing of the building was excellent for the time at which it was designed.*
 - *The building would have lost some stiffness as a consequence of its inelastic deformation.*
 - *There was appreciable non-structural damage to the building.*

2.2 Hotel Grand Chancellor, Christchurch

While the building survived the first major earthquake in 2010, in the February 2011 earthquake, a shear wall located on the ground floor of the Hotel Grand Chancellor failed which nearly led to a disastrous collapse of the building.

The building was originally built as a car park with an office tower above the carpark and later converted to a hotel. The building was supported on piles.

Above the ground, the structure was reinforced concrete. From ground floor to level 14 the structure was of insitu reinforced concrete with flat slabs and cantilever shear walls. The shear walls were irregularly located in plan and not joined.

The extent of the damage was significantly increased by the collapse of a supporting shear wall that failed in a brittle manner because of irregularity in the building (*refer Section 5.7.3*). As a result of the shear wall failure, this was enough to initiate a major stair collapse within the building (*refer Section 5.17*) and failures to columns and beams at various locations. As a result of the stair failure, people had to be airlifted from the top of the building.

The building was subsequently demolished.

2.3 New Royal Adelaide Hospital, Australia

2.3.1 General

The new \$1.85bn Royal Adelaide Hospital is the biggest building/infrastructure project in South Australia's history and when completed will be Australia's newest and most advanced major hospital **Figure 8**.

In the event of a major earthquake in Adelaide, the hospital will be required to provide immediate tertiary clinical care for large numbers of casualties. A critical aspect of the design brief required that the hospital survive such events and retain a high level of operational capacity to deliver post-disaster services.

The majority of the facility has been designed to meet or exceed the requirements for an Importance Level 4 (IL4) structure in accordance with the BCA, together with further technical requirements of the State Government. The serviceability condition for seismic load governed the design.

The 2007 edition of AS1170.4 introduced new performance criteria for Importance Level 4 buildings requiring that they are designed for two distinct earthquake performance criteria discussed later being:

- Life Safety Design
- Serviceability Design

To satisfy these performance requirements, the structural design has focused on controlling storey drift and limiting overall building displacements to ensure

vulnerable non-structural components such as ceilings, services, partitions and the like remain intact and suffer only superficial damage.

It is envisaged that reinforced concrete elements could develop some minor cracking in a structure designed for the serviceability performance criteria but without significant yielding of reinforcement or crushing of concrete. A review of drift limits consistent with this performance standard was undertaken. Eurocode 8, EN1998-1:2004 Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings, recommends a damage limit state for in-plane drift of 0.5% for brittle non-structural elements and 0.75% for ductile non-structural elements. A serviceability drift limit of 0.5% was adopted for this project.

2.3.2 Building sectors and seismic movement joints

The hospital's structural frame is primarily of reinforced concrete, utilising two-way post-tensioned concrete floor plates.

The hospital varies in height from five to ten storeys and covers a footprint of more than 350 x 150 m. The large building footprint necessitated subdivision of the facility into separate smaller independent building sectors. This was done to manage the cumulative long-term movements associated with concrete shrinkage, creep, elastic shortening and thermal effects. Typically, the maximum building length constructible in Adelaide to satisfy these conditions is of the order of 90 m.

Permanent seismic movement joints have been strategically positioned across the hospital footprint so as not to exceed 90 m in any direction while simultaneously achieving the following aims:

- To create floor diaphragms with favourable aspect ratios and inherent structural integrity
- To minimise the overall use of movement joints throughout the facility which are both expensive and problematic from a functional planning perspective
- Where possible, to locate movement joints along partition lines and to avoid high traffic areas.

The seismic movement joints structurally isolate each sector from its neighbours and ensure that lateral earthquake design actions are independently resolved within each sector. The joints are up to 200 mm wide.



Figure 8 – The New Royal Adelaide Hospital under construction 2013

(Photograph courtesy John Woodside)

2.3.3 Structural shear walls

Ductile reinforced concrete structural walls have been adopted as the primary lateral load-resisting system for all building sectors. Such walls have proven themselves repeatedly as providing a reliable method of both limiting drift and preventing structural failure in major earthquakes around the world.

The walls are proportioned and detailed in accordance with AS 3600 Appendix C together with the additional recommendations proposed by Pauly and Priestley⁶, and Priestley et al⁷ to ensure adequate ductility was achieved at calculated limit-state curvatures.

All structural walls and cores are supported on pile footings.

2.3.4 Structural design strategies

The following principles have been incorporated into the structural design to improve inelastic response and prevent collapse:

- Use of redundant lateral and vertical load paths together with continuous edge beams to enhance structural resilience
- Use of direct vertical load paths throughout the building without resorting to the introduction of transfer structures that can create irregularities leading to concentrations of plastic demand and potential collapse
- Preservation of vertical continuity for lateral bracing systems, again avoiding concentration of plastic demand at discontinuities
- Columns proportioned to work comfortably without

requiring the use of high-strength concrete (ie not higher than 50 MPa) and designed with reserve capacity to accommodate drift induced moments and shears arising from both frame action and P-delta effects

- All beam column and slab column joints have continuous anchored bottom face bars passing through the joint to provide post-failure resistance to punching shear via dowel action
- Avoidance of plan irregularities and the adoption of a uniform distribution of lateral bracing elements and mass to control torsional response of each building sector
- Avoidance of ‘soft-storey’ structural performance
- No unreinforced masonry
- Attention to detailing with a focus on the proper anchorage of reinforcement; adequate confinement of concrete in hinge zones and the use of appropriately proportioned boundary elements to structural walls in plastic hinge regions to prevent buckling
- Careful detailing to prevent structural interaction with stiff ‘non-structural’ components such as infill partition walls and precast cladding.

2.3.5 Non-structural elements

In a similar manner, all non-structural elements within the hospital including building services and architectural components have been required to be designed for seismic actions by the specialist subcontractors with all fixings into concrete designed for cracked concrete.

3 Design of buildings

Dr A R Dykes, Institution of Structural Engineers, Scottish Branch, 1978 Chairman's Address, stated that:

"Engineering is the art of moulding materials we do not wholly understand, into shapes we cannot precisely analyse so as to withstand forces we cannot really assess, in such a way that the community at large has no reason to suspect the extent of our ignorance."

There have been significant advances in the past 20 years, in our understanding of how concrete performs under seismic actions, in the technology of concrete, in the design of reinforced concrete structures for seismic actions, with significant changes to reinforcement and enormous advances in computers and software and analysis tools. The computing power and software now available to designers has led to far more elaborate and sophisticated analysis and more refined design of buildings.

This technology can however lull designers into a false sense of security - believing they fully understand how the structure will act under the dynamic actions of an earthquake, when the actual effects from an earthquake may be very different from the computer model. This is because the structure is sized on non-seismic load considerations; seismic hazard and demand used for design may be very different to the ground motions of the real earthquake; member stiffnesses will change during the earthquake as yielding and softening of the reinforced concrete occurs; analysis assumptions greatly simplify real inelastic behaviour; and other factors such as local failures will affect the performance of the structure. This may result in the sophisticated analysis and the design model being inappropriate in a major earthquake event.

3.1 Analysis

Although concrete is a non-linear material and not elastic under ultimate actions, linear elastic analysis is generally used because there have not been any practical or readily available alternatives until quite recently, and elastic methods of analysis have been validated. These elastic methods of analysis are discussed in more detail in the *Reinforced Concrete Design Handbook*²³. The section properties adopted for the structural model can also be complicated and require judgement. Table C.7.1 of the Commentary on AS 1170.4¹⁰ provides some guidance.

The aim of structural design is to produce a safe, serviceable, economical and sustainable structure. Designers should always strive for simplicity, clarity and excellence in their design and detailing and maintain a strong focus on the detailing of reinforcement for seismic actions. Simple design does not mean elementary design but rather well conceived quality design and adequate detailing.

The analysis is only part of the design process. Good designers know there is far more to design than just analysis; they must have a clear understanding of the behaviour of all members, how they are expected to resist the applied actions, and why these members need to be detailed for the seismic actions.

In a real structure, the actual behaviour under load of individual elements is complex; it will depend on the materials used and many other factors, all of which may change under earthquake actions. Idealised computer models of the structure used for the analysis of a structure attempt to simulate how the real structure may behave. Such analyses can be very crude when assessing the structure under real seismic actions.

The analysis and design process is usually a two-stage procedure, viz:

- Structural analysis of the element, frame or structure as a whole to determine member design actions and displacements.
- Analysis of cross-sections of members and the design and detailing of the reinforcement for the cross-sections based on the actions as determined by the structural analysis.

3.2 Steel reinforcement

3.2.1 General

In Australia, reinforcing steel is either Ductility Class L or Ductility Class N where L designates low ductility, and N designates normal ductility in accordance with AS/NZS 4671²⁴. For special buildings, Ductility Class E could be imported from NZ but the structure should be designed and detailed to the NZ Standards.

In Australia, Grade 500 MPa, Class L bars are used to produce fitments and Class L welded mesh incorporating either plain or ribbed bars, may be used as main, shear or secondary reinforcement. Class N bars are either Grade 500 (deformed) or occasionally Grade 250 (plain) bars for fitments. For specific projects, selected reinforcement manufacturers can also produce welded mesh using Class N bars up to N16, in the main direction off coil, in customised sheet sizes.

The main parameters used in AS/NZS 4671²⁴ to define reinforcing steel ductility are uniform strain, A_{gt} , and the ratio of the ultimate tensile strength, R_m , to the yield or proof stress, R_e , obtained from a tensile test. The two ductility classes are specified in Table 2 of AS/NZS 4671 as follows:

Class L: $R_{ek,L} \geq 500$ MPa;
 $R_{ek,U} \leq 750$ MPa;
 $R_m/R_e \geq 1.03$;
 $A_{gt} \geq 1.5$; and

Class N: $R_{ek,L} \geq 500$ MPa;
 $R_{ek,U} \leq 650$ MPa;
 $R_m/R_e \geq 1.08$;
 $A_{gt} \geq 5.0$

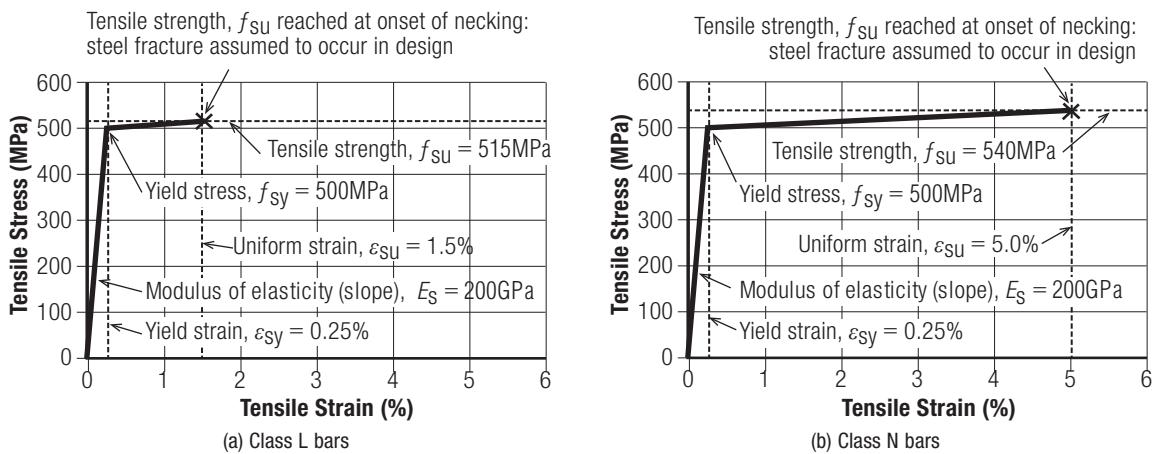


Figure 9 – Design tensile stress-strain curves for Australian reinforcing bars with standard ductility limits

For design purposes, the general stress-strain curves for Class L and Class N reinforcing bars can be idealised using the bi-linear relationships shown in **Figures 9(a) and (b)** respectively, where f_{sy} is the design yield stress ($=R_{ek,L}=500$ MPa), f_{su} is the design tensile strength ($=R_m$, which from above equals $1.03f_{sy}$ or $1.08f_{sy}$ for Class L or Class N reinforcement respectively), and E_s is the modulus of elasticity ($=200$ GPa) before yielding. Yield strain, ϵ_{sy} , equals $f_{sy}/E_s = 500/200000 = 0.25\%$ irrespective of the steel ductility class, and steel fracture is assumed to occur when the strain reaches uniform strain, ϵ_{su} ($=1.5\%$ or 5.0% for Class L or Class N bars respectively) corresponding to the onset of necking at peak tensile force.

Clause 1.1.2 of AS 3600 states that reinforcing steel of Ductility Class L:

“shall not be used in any situation where the reinforcement is required to undergo large plastic deformation under strength limit state conditions.”

Referring to **Figure 9(a)**, this means that in design the Class L steel should not be relied upon to strain more than $\epsilon_{su}=1.5\%$. Similarly, for Class N bars the steel strain should not exceed $\epsilon_{su}=5.0\%$.

Some rules in AS 3600 differentiate between the use of Ductility Class L and N main reinforcement.

For a structural ductility factor $\mu \leq 2$, AS 3600 Appendix C allows the structure to be designed and detailed in accordance with the main body of the Standard and Class L and N reinforcement can be used. Although not covered by AS 3600, any chord members, collector reinforcement or drag bars used in diaphragms (usually floors) must be Class N reinforcement, because of the anchorage requirements and ductility demands on this reinforcement (**Refer Section 5.10**).

Class L main reinforcement can only be welded mesh.

Clause 17.2.1.1 states that:

“Ductility Class L reinforcement shall not be substituted for Ductility Class N reinforcement unless the structure is redesigned.”

which will require the approval and possibly redesign by the designer.

The use of Ductility Class L reinforcement is further qualified by other clauses in AS 3600.

For a structural ductility factor, $2 < \mu \leq 3$, structures have to be designed and detailed in accordance with Appendix C and the main body of AS 3600 provided only Ductility Class N steel is used as flexural reinforcement. Ductility Class L steel is permitted to be used for fitments and non-flexural reinforcement eg shrinkage and temperature.

All welding of reinforcement needs to be in accordance with AS/NZS 1554.3.25

3.2.2 Mesh

Mesh reinforcement is commonly used in slabs on ground and for shrinkage reinforcement and is chosen by some builders/contractors for its speed and ease of construction. It is also used in suspended floors in low rise buildings, such as walk-up units, and occasionally in multi-storey residential structures. It is also used in precast and tilt-up wall panels.

Class N mesh is not readily available, and Class L mesh made from hot-rolled, coiled reinforcing wire of grade D500L complies with the design and detailing requirements of AS 3600 for slabs and walls.

Although Class L reinforcement is allowable under AS 3600 for walls and slabs, where Class L mesh reinforcement is used for loadbearing walls and suspended floors and slabs when acting as diaphragms,

designers need to ensure the reinforcement is capable of meeting the increased ductility demands. For IL4 buildings, mesh is not recommended in structural elements except as shrinkage reinforcement, for secondary reinforcement to steel metal decking or non-structural elements, because of the increased ductility demands.

3.2.3 Steel reinforcement compliance

All steel reinforcing materials must be supplied to conform to the engineering specification and AS/NZS 4671²⁴. The design engineers and builders/contractors must request:

- Third-party, JAS-ANZ product quality certification such as the Australian Certification Authority for Reinforcing and Structural Steels (ACRS) demonstrating compliance to AS/NZS 4671 (for supply) and AS 3600 (for processing), or
- Document quality management systems to the ISO 9000 family of Standards plus Government authority product approvals.

This information is essential to verify the quality assurance and traceability for the issuing of the design compliance certificate for the building project.

3.3 Concrete

While structures will have concrete strengths typically in the range of 25 to 40 MPa, high-strength concrete up to 100 MPa is permitted under AS 3600. High-strength concrete is usually used in columns and walls where the size of such elements needs to be minimised. Care should be exercised when using high-strength concrete in columns and walls for buildings designed for a structural ductility factor $\mu > 2$ or with a post-disaster function. Since high-strength concrete is a brittle material, additional detailing of the reinforcement to prevent brittle failure will be required (*Refer to Section 5*).

Designers also need to nominate on the drawings the construction joints required to ensure that site construction joints are not made where they may affect the seismic capacity of the building.

Where the floor strength is less than 0.75 of the column strength, the implications of Clause 10.8 of AS 3600 for the transmission of axial forces through the floor systems must be assessed early in the design process.

3.4 Load paths

In every building, there are many structural elements used to transmit and resist lateral forces due to earthquake actions. The load paths extend from the top of the building to the footings and can vary from significant structural members to individual elements such as reinforcing bars.

An appreciation of the importance of a complete load path is critical for the designer in understanding how the building might resist seismic actions. Alternative redundant load paths must be considered and designed for wherever practicable.

There are two directions for the primary elements in the load paths, viz those that are vertical or near vertical (columns and walls) and those that are primarily horizontal (floors and roof). The horizontal elements are known as diaphragms, and they collect the lateral forces from each level and distribute them to the vertical elements.

The footings are the last part of the load path. They transmit the lateral forces to the foundation material, support the building and resist lateral and overturning forces.

In addition to the primary load path and primary elements, there are many secondary elements that must also be adequately connected to carry the earthquake actions throughout the structure. The failure of a primary or secondary element will usually result in additional actions being placed on adjoining elements. If the adjoining elements are unable to take these additional demands, progressive failure or collapse may occur.

3.5 Robustness

Bill Boyce, former Associate Professor Civil Engineering, the University of Queensland stated that:

"We had a discussion stressing the need to provide robustness in structures to give them a better chance of resisting earthquakes - effective load paths, design for ductility and careful detailing to accommodate the disaster scenario. It is not always difficult or expensive to improve robustness provided a reasonable structural system is chosen in the first place. Improved robustness, in my opinion, is more valuable than larger design forces."

Structural robustness is discussed briefly in the commentary to AS 1170.0 but is not well defined. There are no specific requirements for design for structural integrity (the prevention of progressive collapse) or robustness in the BCA or AS 3600. The AS 3600 Commentary²⁶ has some limited information on this requirement. Nevertheless, because of overseas experience and failures, designers must consider the robustness of reinforced concrete buildings including reinforcement detailing.

The *Ronan Point* failure in the UK in 1968 occurred when a block of flats incorporating structural precast concrete wall panels failed because a relatively minor gas explosion blew out the loadbearing precast walls on one corner of the building. This failure led to concerns about robustness and, as a result, both the UK and Europe introduced specific requirements for robustness, including horizontal tying of the structure at each floor.

In recent times, the US has considered progressive collapse with failures such as the bomb attack on the Alfred P. Murrah Federal Building in 1995 and the collapse of the towers at the World Trade Centre in New York in 2001. More information on these failures can be found in the paper by Gene Corley in Concrete 2005²⁷.

In simple terms, a structure ‘should be safe’. The Eurocode provides the following definition of robustness:

“the structure shall be designed and executed in such a way that it will not be damaged by events like fire, explosion, impact, or the consequences of human error, without being damaged to an extent disproportionate to the original cause.

Progressive and disproportionate collapse must be avoided at all times. This means that the failure of one member should not set off a chain of events where the structure progressively collapses as occurred in the failure of a row of columns in the Newcastle Workers Club in 1989 which is thought to have initiated the total collapse of the building¹³.

Robustness considerations require that all structures have an inherent resistance to lateral loadings, and if none is specified, then a notional percentage of the vertical actions should be adopted. Redundancy also becomes an important design issue to avoid the failure of any single loadbearing member leading to the collapse of the entire structure.

The building structural form will significantly affect its robustness and, therefore, needs to be considered at the concept stage. An example of this might be a large transfer beam supporting a large part of the building; failure of this element would be catastrophic and should be avoided if possible or the design robust enough to provide a considerable reserve of strength. Often such members are designed with an additional load factor, or sized to remain elastic under earthquake design actions.

Buildings should have sufficient robustness to survive without collapse if subjected to ground motions in excess of that specified by the Standards. Well-proportioned and well-detailed insitu reinforced concrete structures are inherently robust. It is important to ensure that the structure is tied together, can resist the applied lateral actions and that the failure of a particular element will not lead to progressive collapse. In the case of tilt-up and precast concrete structures, the provision for robustness may require more design consideration.

Reinforcement detailing for robustness also needs to address some of the basic requirements such as:

- Minimum reinforcement should be provided in both faces of horizontal members such as beams and floor slabs at supports even if the design does not require it or detailing is not required by the Standard. For example, properly anchored bottom face reinforcement should be provided in horizontal members over columns and support walls for continuous members.
- Detailing in accordance with Appendix C of AS 3600 is required for buildings with a post-disaster function and for buildings where the ductility $\mu > 2$.
- Critical members should be reviewed for their role in the structure, detailed as required, and alternative load paths considered.

Further information can be found in *Practical guide to structural robustness and disproportionate collapse in buildings*²⁸ and *A Review of Progressive Collapse Research and Regulations*²⁹, both of which highlight the importance of tying members together, the need for alternative load paths and adequate design resistance.

3.6 Acceptable drift limits

AS 1170.4 sets out the maximum drift requirements for the design methods set out in Sections 5 and 6 of the Standard. However, the maximum inter-storey drift must not exceed 1.5% of the storey height at each level at the ultimate limit state with potential reduced stiffnesses. These maximum drifts, however, are large lateral displacements of the order of 30 to 50 mm between typical floors. Many structures may not be able to accommodate such drifts without premature failure of structural elements. Also, calculations associated with drift are often poorly understood, and stiffness assumptions are sometimes wrong.

Designers may need to specify lower drift limits for flexible structures or special structures such as hospitals or buildings with special design requirements. Structures, which are EDC 1, do not have to be designed for drift as they are not more than 12 m in height.

Even if a part of a structure is not designed specifically to withstand seismic forces, it must be designed for the full drift (deflection) of the whole structure or calculated in accordance with Clause 5.4.2, 5.5.4 or 6.7.1 of AS 1170.4. Moment-resisting frame systems are much more flexible than shear wall systems and need a careful review for drift, especially with associated shear walls. The commentary on AS 1170.4 suggests that for EDC II structures where the primary seismic-force-resisting elements are shear walls extending to the base, the drift is expected to be small.

It is very important that non-structural components such as facade systems, services and partitions and ceilings be designed and detailed to accommodate the anticipated maximum drift. Lower bound estimates of structural stiffness should be used in such drift calculations.

3.7 Ductility demands

One of the issues when designing structures in an area of low seismicity such as Australia is that when a major earthquake occurs which exceeds the design return period (annual probability of exceedance of 1/500 or 1/1000), then the increase in peak ground acceleration over the design event can be significant; and therefore the increase in the lateral forces can be large. For a rare event with an annual probability of exceedance of 1/2500, this increase in demand can be of the order of 3 times that for a low seismicity risk area. This increase is shown diagrammatically in Figure 10. However, for structures in areas of high seismicity, the increase in peak ground acceleration is much less significant, perhaps only about 30%.

Geoscience Australia’s 2012 isothermal maps, suggests a hazard value, Z, of 0.06 for Melbourne and Sydney for a

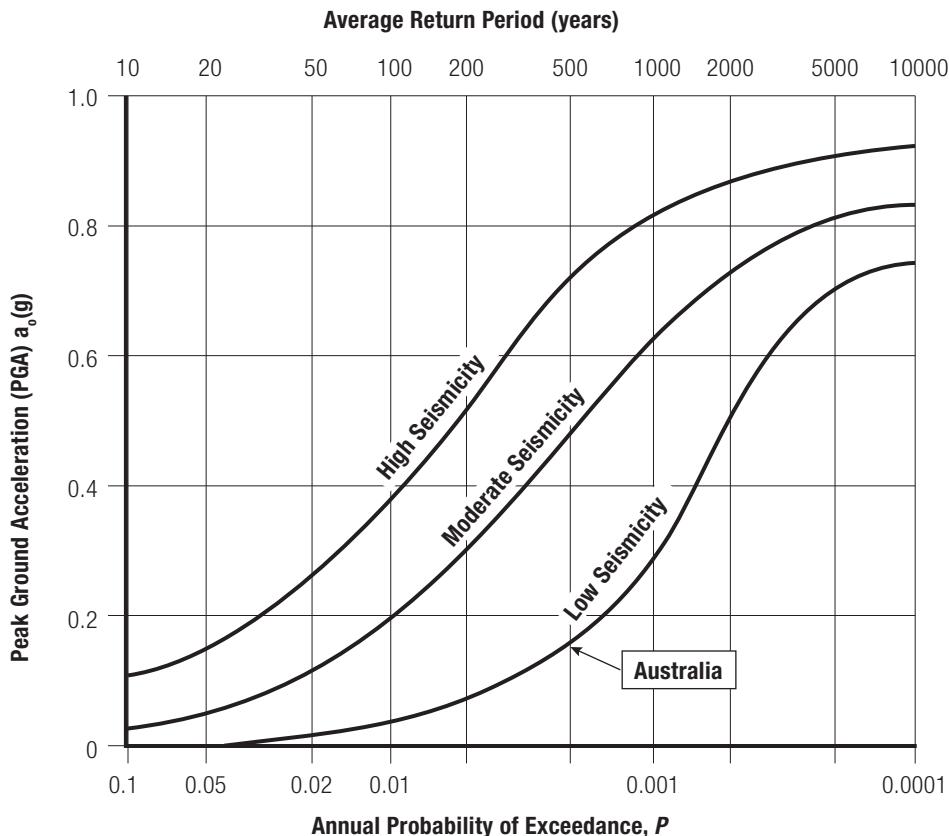


Figure 10 – Average annual probability of exceedance and the effect on peak ground acceleration
(From Paulay & Priestley⁶)

500-year event and about 0.16 for a 2500-year event, ie about a threefold increase in ground motion similar to that shown in Figure 10.

The consequence of this issue is that structures designed for a 500 year event in areas of high seismicity are quite likely to exhibit satisfactory performance during a much rarer event. However in Australia, for a 500 year ground motion, structures are unlikely to survive a rare but foreseeable larger event with much longer return periods.

3.8 Special study for Importance Level 4 structures

AS 1170.4 requires a special study for Importance Level 4 (IL4) structures to be carried out to demonstrate that they remain serviceable for immediate use following the design event associated with an Importance Level 2 structure. An IL4 structure is, therefore, designed for two distinct earthquake events as follows.

3.8.1 Serviceability design earthquake

The BCA requires IL4 structures to remain serviceable after a design earthquake event that has an annual probability of exceedance of 1/500. The serviceability earthquake models a statistically more frequent event with less intense ground motion, after which the building is required to remain operational immediately. During such an event, some minor damage is acceptable provided it is easily repairable and would not interfere

with the ongoing operation of the building, and essentially the building should perform elastically.

To satisfy this performance requirement, the structural design should focus on controlling storey drift and limiting the overall building displacements to ensure vulnerable non-structural components such as ceilings, services, partitions and the like remain intact and suffer only superficial damage. It is suggested that these drift limits should be of the order of 0.5 to 0.75% of the storey height.

It is envisaged that reinforced concrete elements could develop some minor cracking during the serviceability earthquake but without significant yielding of reinforcement or crushing of the concrete.

3.8.2 Life safety design earthquake

The BCA requires IL4 structures to provide life safety after a design earthquake event that has an annual probability of exceedance of 1/1500. This design event simulates a major earthquake with an annual probability of exceedance set by the BCA of 1/1500. That is a low-probability, high consequence event for which the primary design objective is to preserve the lives of building occupants and those near the structure. It is envisaged that both the structure and its contents will suffer significant damage during such a severe event, but collapse is prevented. Design strategies need to ensure these design objectives are met for the structure and parts and components.

4 Earthquake design

4.1 Methodology

“

A chain is only as strong as its weakest link ...

If one considers a chain comprising a number of links, each with equal strength and ductility, and subjects it to a tensile force, which link will yield first and then break first?

The answer is that it is not possible to predict which link will yield first and indeed which one will fail. The structural engineer is in a similar predicament regarding the seismic performance of a structure. The point of yield and then failure could be at any one of many locations, affected not just by such considerations as design strength and stiffness, but also by such matters as the provision of a greater number of bars than required because of rationalisation of bar size or spacing in the design, strain-hardening of reinforcement during cyclical action, and the over strength of both the concrete and reinforcement material above their specified compressive strength or nominal yield strength respectively, and the direction and intensity of the earthquake. What is important is not where yield first occurs, but that our structures are designed with sufficient robustness and redundancy to prevent their collapse.

4.2 Design methods

4.2.1 Force-based seismic design method

Traditionally, earthquake design has been based on a **quasi-static** forces approach where **hypothetical** static actions are applied to simulate the inelastic dynamic response to an earthquake. Unfortunately, this may bear no resemblance to the forces that occur, as earthquakes do not know about Standards, methods of design or indeed the building being designed. Designers should note that the information in this Guide is based on the force-based design method used in AS 1170.4. Other methods of design are discussed to alert the designer to some of the issues and likely future changes to the Standards.

The fundamental problems with the force-based method of analysis include choosing the right model, the selection of appropriate member stiffness and whether or not the static forces are appropriate for the design being considered. Member stiffness assumptions that are used to determine both the building and the internal distribution of actions, cannot be resolved until the design is complete, and they will change during a seismic event as the structure cracks and yields.

As static design actions (forces) are at best crude approximations of actual behaviour, sophisticated analysis is unwarranted and provides designers with only a general overview of what might happen. It must be remembered that the actual actions may be significantly different during a real earthquake.

4.2.2 Capacity-based seismic design method

In response to problems with the force-based design method, the capacity design method was introduced in New Zealand and is discussed in Paulay and Priestley⁶. Structural failure occurs when the strength of the member is exceeded by the forces placed upon it, ie, 'demand' exceeds 'capacity'. This capacity then is set at the 'ultimate limit state' or the maximum credible earthquake the structure may be expected to undergo during its life. Below this, the structure would be expected to survive without experiencing severe damage.

Capacity design involves the designer choosing the positions of failure in the structure and ensuring that at these locations, the failures are ductile and controlled in a manner so that they are not brittle or catastrophic. Further, by selecting the location of the formation of these ductile failures or '*plastic hinges*', the designer ensures that failures do not occur at other critical points in the structure. Failure in the structure outside the '*plastic hinges*' is prevented if member strength is in excess of the maximum strength (including over strength) of the ductile elements or '*plastic hinges*'.

4.2.3 Displacement-based method

As discussed earlier, traditional force-based design is based on the calculation of forces, the determination of which rely on estimates of building stiffness and period. A number of serious problems are known to exist with this approach, all of which are well documented and discussed by many authors including Priestley et al⁷. Key difficulties associated with force based design are known to include:

- The need to estimate element size and stiffness before undertaking analysis and design. The stiffness of each component is difficult, if not impossible, to calculate accurately and will change during the earthquake
- The assumption that element stiffness is independent of strength for a given section
- The use of blanket ductility factors to assess structural systems containing multiple lateral load paths with widely varying ductility demands and capacities.

It is beyond the scope of this Guide to discuss in detail the reasons why these issues undermine the current force-based approach, however the interested reader is again referred to Priestley, et al⁷; Wilson and Lam³⁰, and Wilson and Lam³¹. Since the early 1990's, deficiencies in forced-based design methods have led to research into alternative design strategies which have increasingly recognised the importance of deformation as a more appropriate indicator of structural performance rather than simple strength. For example, a heavily reinforced column that is also heavily loaded will have less ability to inelastically accommodate drift deformation than a column of the same size which is lightly loaded and reinforced. Failure to recognise this can lead to premature failure of critical structural elements. A

number of displacement-based design methods now exist which avoid the above problematic assumptions.

As a simple entry into this area, it is recommended that the reader review the works of Wilson and Lam which explain in general terms how displacement-based design principles can be applied in the Australian context. These methods are permitted by the current Australian Standard and once understood can be simpler to apply than traditional forced-based methods. The Capacity Spectrum method outlined by Wilson and Lam compares the Structural Capacity curve determined from a pushover analysis to the site specific Acceleration – Displacement Response Spectra (ADRS) Demand curve **Figure 11**. The intersection of these two curves identifies the system design acceleration and displacement and indicates a safe design. The procedure for transforming the Acceleration Response Spectra found in AS 1170.4 into an equivalent ADRS is explained in the Australian Earthquake Engineering Society (AEES) Commentary to the Standard, whilst simplified methods for determining the capacity curves for columns and walls have recently been published by Wilson et al³².

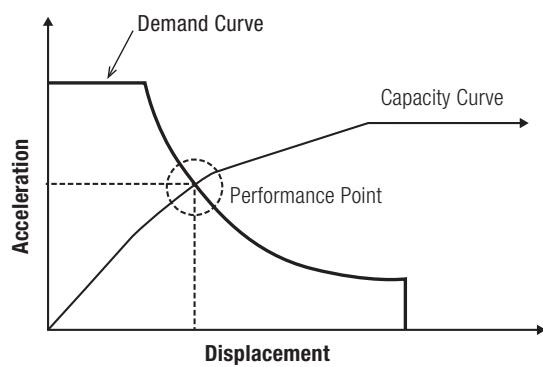


Figure 11 – Capacity spectrum method

(Source Australian Journal Structural Engineering³²)

5 Structural systems

5.1 General

Structural systems should be as simple as possible and have clear readily understandable gravity and lateral stability load paths. Some structural systems perform better than others in resisting earthquake-induced forces. One of the early tasks of the structural designer is to select an appropriate structural system that achieves the best system for seismic performance for building within the constraints dictated by the architect, the site and other restraints and conditions. Alternative structural configurations should be considered at the concept stage to ensure that an undesirable geometry and structural form is avoided or minimised before the detailed design of the building begins **Figure 12**. In particular, structural irregularities both vertically and horizontally (discussed later) must be considered early in the design phase, and sound structural engineering principles applied to avoid or mitigate these effects.

Many multi-storey reinforced concrete buildings built in Australia will be in either Earthquake Design Categories II or III designed to AS 1170.4, depending on the ground conditions and will be below 25 or 50 m in height.

AS 1170.4 requires that all parts of a structure be interconnected, in both the horizontal and vertical directions. Connections between structural elements are

typically the weakest link in the chain and should be detailed to fail in a ductile manner to avoid rapid degradation of strength under earthquake actions. Connections must be capable of transmitting the calculated horizontal earthquake design actions to maintain load path integrity throughout the structure and for the earthquake forces to be transferred to the footings and foundation. In turn, the foundation must be robust enough to accommodate the overload due to a significant event without catastrophic loss of strength.

Diaphragm action must be considered in the design of the floors to provide the connection between the structural walls and columns and the roof and floors to transfer forces in and out of these elements. The provision of adequate anchorage between floor diaphragms and structural walls and columns is of particular importance. Openings in shear walls and diaphragms also need additional reinforcement at the edges and corners to resist local stresses (*Refer Section 5.10*).

In Australia, stair and lift cores are typically constructed with reinforced concrete walls because of fire rating requirements and industry preference. As a result, most buildings in Australia will be either a concrete shear wall system or a combination of concrete shear walls and moment-resisting frames or moment-resisting frame.

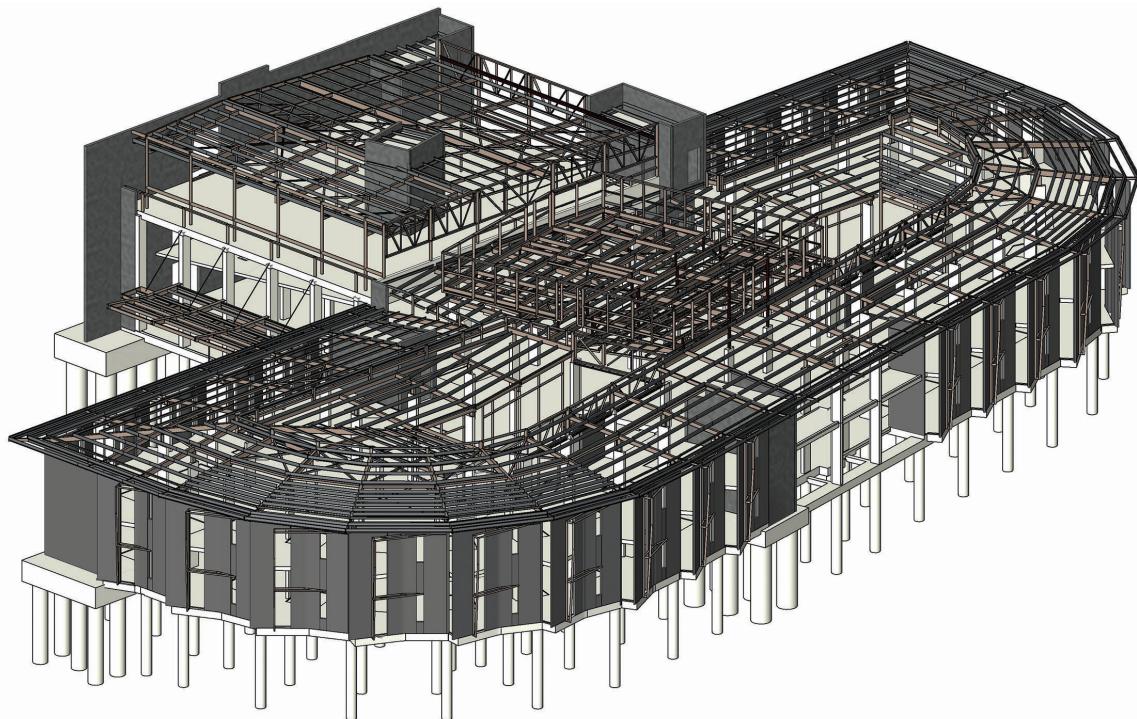


Figure 12 – 3D view of an irregular structure

(Note: Most of the building is two storeys high and the view shows the piles in the ground supporting the building)

(Drawing courtesy Wallbridge & Gilbert)

Some low-rise structures may be moment-resisting frames only. The designer has to choose whether the shear walls are to be designed as ductile or of limited ductility. Once the structural system is chosen, the structural ductility factor, μ , and structural performance factor, S_p , can be determined in accordance with Table 6.5 (A) in AS 1170.4 or Table C3 in AS 3600 or **Table 1** of this Guide. Ductile shear walls are often chosen where earthquake forces are higher than wind forces as the seismic reduction factor will lead to smaller members, particularly footings. The decision as to which design route to take is left to the designer.

If the designer chooses an Ordinary Moment-Resisting Frame (OMRF), then the ratio of Structural Ductility Factor, μ , to the Structural Performance Factor, S_p , will result in the earthquake design actions being increased by about 73% over those for an Intermediate Moment-Resisting Frame (IMRF). OMRF's are deemed to require no further detailing consideration beyond the detailing required in the body of AS 3600 (*Refer Section 5.6.3 for recommended minimum detailing requirements and the strength of beam column joints*).

The choice of an IMRF considerably reduces the horizontal earthquake base shear force because of the extra ductility provided by the improved detailing. The improved detailing will often prove economical by allowing reduced member sizes or permitting a higher level of earthquake resistance.

Modelling of more sophisticated structures, particularly those with basement levels, requires careful attention. Building complexes with two or more towers on the same common podium can also result in significant design issues, particularly within the podium diaphragm connecting the towers which will move separately and tear the podium apart. The separation of the seismic effects of such towers often requires the introduction of seismic control joints between the towers.

One problem with moment-resisting frames (MRF's) is their lack of lateral stiffness and the large displacements (drift) they experience under earthquake actions - often together with incompatibility of the rest of the structure in resisting such drifts. This can result in significant damage to adjoining structural elements and non-structural parts and components. To avoid collapse, it is important in MRF's that plastic hinges form in the band beams and not the columns in an extreme event. Where band beams are used, they are significantly stiffer than the columns, often making this goal difficult to achieve (*Refer Section 5.6*).

Where excess strength is provided above that theoretically required by the design (through rationalising the design), less ductility is required for the element, eg due to the provision of additional reinforcement, or extra thickness or depth of section for fire or deflection requirements. Therefore, less seismic detailing may be satisfactory and improve buildability. This approach however needs to be adopted cautiously, as providing over strength to one part of the structure in isolation may inadvertently increase the demands elsewhere, resulting in premature failure and collapse.

5.2 Detailing and drawings of concrete elements

Structural analysis including the sizing of the elements of the structure are the first part of the overall design process of a structure; detailing and drafting of the drawings the second part. Detailing and drafting consists of preparing adequate plans, elevations, sections and details and requires an understanding of how each part of the structure will perform under seismic actions.

Detailing of the reinforcement is a vital part of the seismic design process for reinforced concrete. Detailing must provide sufficient transverse steel to prevent shear or crushing failures, develop the anchorage of reinforcement into areas of confined concrete and prevent the buckling of compression steel, once the cover to the concrete has been lost due to spalling as a result of cyclic movements. Main bars must not lose their anchorage into the surrounding concrete during the repeated reversing load cycles that they will experience during a major earthquake. Reinforcement anchorage lengths connecting various individual parts of the structure together must be sufficient and allow for local failures without collapse.

The art of reinforcement detailing is to provide reinforcement in the right places required by the design and to meet the expected earthquake demands. If the reinforcement is correctly designed, placed and fixed and the concrete correctly placed around the reinforcement, then the structure will comply with the intent of the design and should perform satisfactorily during its design life, including under seismic actions.

Detailing involves practical and thorough consideration on how and where the reinforcement should be placed. Experienced designers who understand the overall design and the seismic requirements of the building should be responsible for the overall detailing. Detailing must not be carried out by graduate, inexperienced engineers or drafters without senior supervision.

In any reinforced concrete design, it is necessary that the designer accepts responsibility to detail clearly and specify the reinforcement requirements on the drawings.

Adequate time must be allowed to complete the structural design and to detail the reinforcement adequately for all concrete elements, together with suitable checking and coordination. Checking should occur prior to the issue of the drawings for construction and manufacture of reinforcement.

The detailing requirements of AS 3600 generally follow those of ACI 318M-14¹⁸. With the trend to off-site prefabrication of reinforcement, attention needs to be given by designers as to how the individual reinforcement components can be prefabricated and joined on site by dropping in splice bars (known as loose bar detailing). Designers should refer to the CIA Reinforced Detailing Handbook for Reinforced and Prestressed Concrete for this detailing³³.

5.3 Regular and irregular buildings

The first edition of AS 1170.4 included a section on the configuration of the building, including whether the building was regular or irregular, which affected the design requirements. The second edition does not include this requirement, now assuming that all buildings are irregular. NZS 1170.5 has a section on irregularity³⁴. For example, the building in **Figure 12** is irregular.

The configuration of a structure can significantly affect its performance during a strong earthquake, with regular arrangements being superior. Configuration can be divided into two types, plan configuration, and vertical configuration. Past earthquakes have repeatedly shown that structures that have irregular configurations suffer far greater damage than structures having regular configurations.

Structures should have regular configurations wherever possible, recognising that many buildings must suit the site or other constraints including the architectural layout and that a regular configuration may not be possible.

While modern analysis programs may allow for the design of building irregularities, the designer must study the building structure from the beginning of the design process and understand where the structure might fail because of its irregularity. Earthquakes have consistently shown that significant irregularity can lead to failure and greater damage, even with more sophisticated analysis, design, detailing and construction. This fact was once again confirmed in Christchurch in 2011.

5.3.1 Plan configuration

- **Torsional irregularity.** A structure may have symmetrical geometric shape without re-entrant corners or the like but still be classified as irregular in plan because of the distribution of mass or vertical earthquake-resisting elements where the centre of mass and the centre of stiffness (shear centre) are significantly different, eg an offset core at the side of the building.
- **Re-entrant corners** **Figure 13.** A structure having a regular configuration can be square, rectangular, or circular. A square or rectangular structure with minor re-entrant corners might still be considered regular, but large re-entrant corners would be classified as an irregular configuration. The response of such irregular structures is usually different from the response of a regular structure of the same overall dimensions, and they will be subject to higher local forces than would be determined by the use of the Standard without modification due to torsional moments.

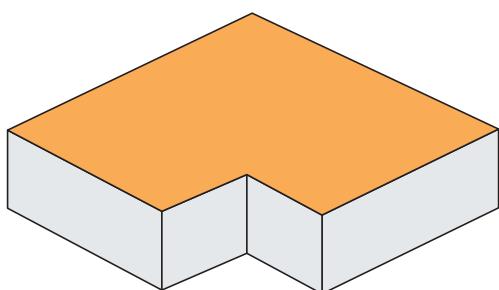


Figure 13 – Re-entrant corner irregularity

- **Diaphragm discontinuity** **Figure 14.** Significant differences in stiffness between parts of a diaphragm at a particular level due to discontinuity or openings are classified as irregularities since they may cause a change in the distribution of horizontal earthquake forces to the vertical components and create torsional forces not accounted for in the normal distribution considered for a regular structure, eg large floor penetrations such as a floor atrium.

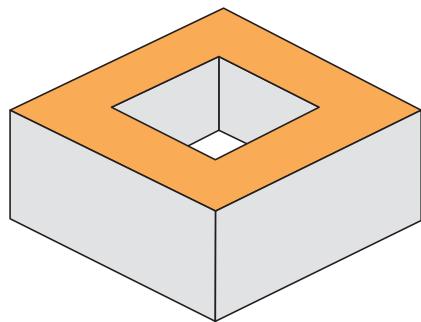


Figure 14 – Diaphragm discontinuity irregularity

- **Out-of-plane offsets** **Figure 15.** Discontinuity in horizontal load paths such as offsets in vertical elements creates a structure that can no longer be considered regular. The most critical of the discontinuities is the out-of-plane offset of vertical elements of the horizontal earthquake-force-resisting elements. Such offsets impose vertical and horizontal demands on horizontal elements that are difficult to provide for adequately, eg transfer beams.

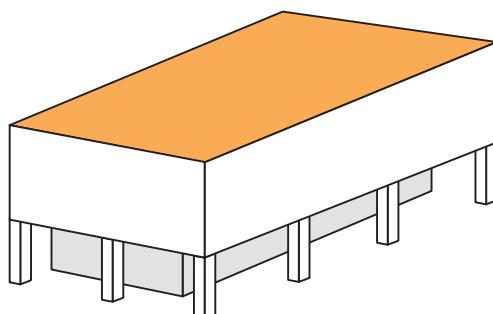


Figure 15 – Out-of-plane offset irregularity

- **Non-parallel systems.** Where vertical elements of the horizontal earthquake-force-resisting system are not parallel to, or symmetrical with respect to, major orthogonal axes, the static horizontal force procedures cannot be applied as given; the structure should thus be considered to be irregular, eg structures framed on a radial grid arrangement that is not a full circle.

5.3.2 Vertical configuration

Vertical configuration irregularities affect the response of the building under seismic actions at the various levels of the building and will induce actions at these levels that can differ significantly from the distribution assumed in the analysis.

- **Stiffness irregularity** (soft storey) **Figure 16**. This might occur where the horizontal stiffness of the storey is significantly less than the storeys above or below the one being considered. An example of this might be a moment frame at ground floor and braced shear walls or shear walls above, or significantly different floor-to-floor heights.

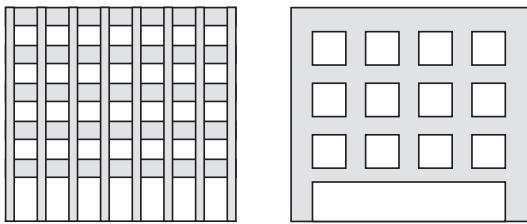


Figure 16 – Examples of buildings with a soft first storey, a common type of stiffness irregularity

- **Strength irregularity** (weak storey). A structure is classified as irregular if the ratio of mass to stiffness in adjoining storeys differs significantly. This might occur when a heavy mass, such as a swimming pool, is located at one level.
- **Gravity load irregularity**. This might occur where one storey is taking a significantly higher gravity load than the floors above or below, eg plant room.
- **Vertical geometrical irregularity**. The structure may have a geometry that is symmetrical about the vertical axis and still be classified as irregular because of significant horizontal offsets in the vertical elements of the horizontal earthquake-force-resisting system at one or more levels.
- **In-plane discontinuity irregularity in the vertical plane of horizontal force-resisting elements**. The problem of concentration of energy demand in the resisting elements in a storey as a result of abrupt changes in the strength capacity between storeys has been observed in past earthquakes **Figure 17**.

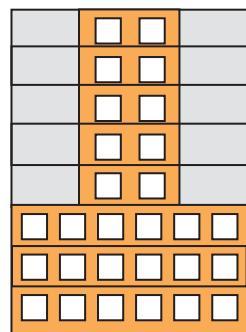


Figure 17 – Example of in-plane discontinuity irregularity

5.3.3 Torsional Effects

Horizontal earthquake actions are assumed to act through the centre of mass of the structure. Torsion in structures results when the centre of stiffness of the walls and columns (often referred to as the centre of resistance, or point about which the structure resists the lateral actions including earthquake) does not correspond to the centre of mass of the structure and results in torsion actions being resisted by the walls and columns.

When calculating the seismic actions using AS 1170.4 it requires that an additional 'accidental' eccentricity be included as outlined in Clause 6.6 of AS 3700 as follows:

“For each required direction of earthquake action, the earthquake actions, as determined in Clause 6.3, shall be applied at the position calculated as $\pm 0.1b$ from the nominal centre of mass, where b is the plan dimension of the structure at right angles to the direction of the action.”

This $\pm 0.1b$ eccentricity shall be applied in the same direction at all levels and orientated to produce the most adverse moment for the 100% and 30% loads.

This 'accidental' eccentricity allows for items such as building services, partitions, storage units, furniture and stairs, which are either difficult to account for or unknown at the time of the design.

How is this Clause interpreted? For symmetric structures: in the direction of the earthquake load, 100% of the earthquake force is offset by a distance of $0.1b$ from the centre of mass, and then in the perpendicular direction, 30% of the earthquake force is applied, also offset from the centre of mass by a distance of $0.1b$, where b is the width of the building in that direction. For asymmetric or torsionally unbalanced structures, this 'accidental' eccentricity must be added to the static eccentricity or distance from the centre of stiffness (or resistance) to the centre of mass.

Thus, the shear force in a wall or column has two components: direct or translational shear (no torsional effects) and torsional shear from 'accidental' eccentricity or static plus 'accidental' eccentricity. Both the earthquake shear force and torsion must, therefore, be distributed amongst the shear resisting elements in proportion to their stiffnesses.

The forces induced by the earthquake at each level are used to calculate the torsional moment at that level. To calculate the design shear action on ground floor elements, if the structure is uniform for its entire height, the base shear can be used to calculate the total torsional moment to be distributed to the various shear-resisting elements. If the floor plan dimensions or the distance from the centre of mass to the centre of stiffness varies, then the torsion at each floor level will need to be calculated and added to obtain the total torsion on the building.

5.4 Earthquake Design Category (EDC)

AS 1170.4 specifies three levels of earthquake design for Importance Level 2, 3 and 4 buildings (excluding domestic construction less than 8.5 m in height – Refer Appendix A of AS 1170.4), viz:

- EDC 1 applies only for structures ≤ 12 m in height. This design category would apply only to a limited number of concrete buildings. It involves static design using a simple equation for lateral actions and there is no requirement for drift.
- EDC 2 applies only for structures > 12 m or under 25 m or 50 m in height depending on the Importance Level. This design category will apply for many concrete buildings in Australia. It involves static design although dynamic analysis is permitted and commonly used. For buildings less than 15 m in height, simplified static design can be used.
- EDC 3 applies only for structures over 25 or 50 m in height depending on the Importance Level. This design category would apply for larger buildings in Australia and requires a dynamic analysis for the building.

AS 1170.4 also allows the designer to use a higher EDC value than that required by the Standard.

5.5 Joints and pounding

Movement during seismic events must also be considered at the movement joints in the building. Such joints must be sized to accommodate the seismic movements, together with thermal, shrinkage or other design movements as appropriate. Movement can be perpendicular or parallel to the joint. These joints need to be correctly detailed and may require proprietary joints as shown in **Figure 18**. Joint details will include the floors, walls, ceilings and finishes. Joints should not go through critical areas, and architectural parts, components and building services crossing these joints need to be detailed to allow for the total expected movements, which can be very large.

Pounding or the impact of adjoining buildings due to different dynamic earthquake responses can be a serious problem and one that unfortunately is sometimes overlooked. Pounding can be critical in large cities due to the close proximity of many tall buildings with different periods of vibration. The damaging effects of this situation were observed during the Mexico Earthquake in 1985.

AS 1170.4 requires that structures over 15 m in height be separated from adjacent structures or set back from the building boundary by a sufficient distance to avoid damaging contact. This is deemed to comply if the primary seismic-force-resisting elements of the structure are walls extending to the base for Earthquake Design Category II or the setback from the boundary is more than 1% of the structural height in other cases. For a tall building, this can be a significant distance. For irregular buildings, especially those with moment-resisting

frames, considerable torsional twisting in plan and significant drift can often occur and this needs to be carefully considered during design.

5.6 Moment-Resisting Frame systems (MRF)

Moment-resisting frames are essentially complete space frames that support vertical and horizontal actions by both flexural and axial resistance by the members and connections.

There are three types of moment-resisting frames: ordinary, intermediate and special. The difference is the degree of ductility offered by the reinforcing design and detailing. Table 6.5(A) of AS 1170.4 generally assigns a structural ductility factor μ of 2 to ordinary, 3 to intermediate and 4 to special moment-resisting frames. Increased structural ductility means that more of the earthquake action can be carried in the ductile or plastic regions of the structure. However, structures are rarely only a moment-resisting frame system as they often include concrete walls for stairs and lift shafts and therefore, are a combined system and typically assigned a structural ductility factor of 2. Only low-rise buildings are likely to be entirely moment-resisting frame systems where the stair and lift shafts are of lightweight construction, isolated and not designed for lateral actions as shear walls.

5.6.1 Ordinary Moment-Resisting Frames (OMRF)

OMRF's need no specific reinforcement detailing for seismic resistance. Detailing is set out in the main body of AS 3600 and higher earthquake design forces consequent from the use of lower structural ductility factor, $\mu \leq 2$ and higher structural performance factor, S_p are required by AS 1170.4.

The designer should note that for OMRF, detailing to the body of AS 3600 is likely to result in only limited frame ductility, primarily as a result of the poor beam column joint performance in the event of an earthquake. Joint failure may result in the collapse of the building (Kobe, Northridge, Mexico City, and Newcastle).

Although not required in AS 3600, in low-risk seismic areas in the USA similar to that of Australia, ACI 318 requires for an OMRF that at least two longitudinal bars are continuous along the top and bottom faces of the beam members to provide additional ductility. Also for columns with a clear height dimension of ≤ 5 m, ACI 318 requires that the columns be designed for shear in accordance with requirements for an intermediate moment-resisting frame (IMRF).

It is important to ensure that an OMRF is sufficiently robust to cater for forces it may experience during an earthquake larger than the one assumed in design. The designer must detail the OMRF with care to ensure that plastic hinges do not form at the locations shown in **Figure 18 (a)** forming what is called a soft storey. The system as shown in **Figure 18 (b)** is the most desirable result.

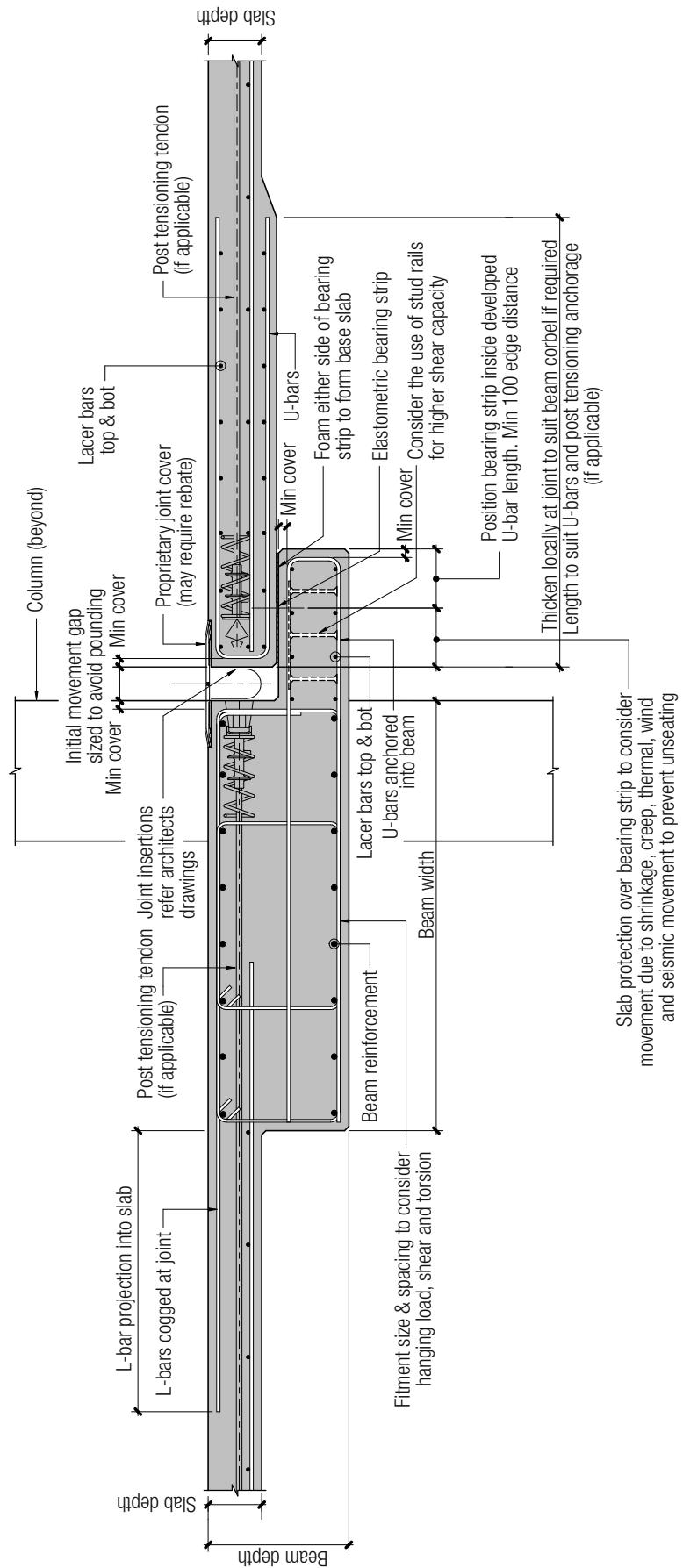


Figure 18 – Typical corbel arrangement

To achieve the required ductility, detailing should be provided so that plastic hinges can form in the beams. (Refer Section 5.6.2 Intermediate Moment-Resisting Frames.)

AS 3600, including Appendix C, unfortunately does not specifically direct the designer to provide a weak beam/strong column mechanism **Figure 19 (b)** so any of the three indicated sidesway mechanisms in **Figure 19** could occur during a seismic event of sufficient magnitude to cause yielding of reinforcement.

5.6.2 Intermediate Moment-Resisting Frames (IMRF)

An intermediate moment-resisting frame is a moment-resisting frame which can be regarded as achieving a structural ductility factor of up to 3 if the additional reinforcement detailing requirements of Clause C4 of AS 3600 are adopted. Because of the detailing and hence higher structural ductility factor, they are designed for lesser seismic loads than for an ordinary moment-resisting frame. Note that the beam-column joints need to be appropriately considered and detailed, preferably to provide a strong column/weak beam configuration. (Refer Section 5.14.4 Beam column joints).

5.6.3 Special Moment-resisting Frames (SMRF)

A special moment-resisting frame is a moment resisting frame with extra detailing over an intermediate moment-resisting frame and, therefore increased ductility ($\mu > 3$), and reduced seismic actions. They will rarely be used in Australia and should not be used for IL4 structures because of the excessive drifts associated with frame action. As the design of structures with $\mu > 3$ is outside the scope of the standard, AS 3600 refers designers to NZS 1170.5³⁴ or designers could use ACI 318M-14¹⁸ for the design of such frames and a National Earthquake Hazard Reduction Program (NEHRP) document³⁶ also provides guidance on detailing of special moment-resisting frames.

5.7 Walls

5.7.1 General

Concrete walls are typically used to resist both vertical and horizontal actions in buildings. They can be constructed of either insitu concrete, tilt-up concrete or precast concrete and will have different functions within a building. They can act as an element supporting vertical actions, as a slab resisting horizontal actions, as a vertical cantilever in flexure (ie a shear or structural wall) acting as a deep beam, or more commonly a combination of these functions. The failure of shear walls under seismic actions will usually lead to the collapse of the building.

Frequently a structural core comprised of reinforced concrete walls will be assumed to take all the lateral actions. However the contribution of other secondary walls and in particular all loadbearing precast concrete walls must not be ignored as they will contribute to the lateral resistance of the structure and may be more critical because of drift which is frequently a structural requirement. Secondary walls will attract lateral load and may prove to be more critical than the primary core due to their limited drift capacity.

In the 1994 edition of AS 3600, the design of walls changed from ACI 318 to BS 8110³⁷ simplified rules. AS 3600 suggests that the rules for the design of walls applied only to a restricted set of walls and therefore designers need to be careful about pushing the limits of the design of walls using the simplified design methods when axial actions are high. Particular care needs to be exercised in the detailing of centrally reinforced walls designed using the simplified method that have the potential to experience tension during earthquakes that may exceed the design action.

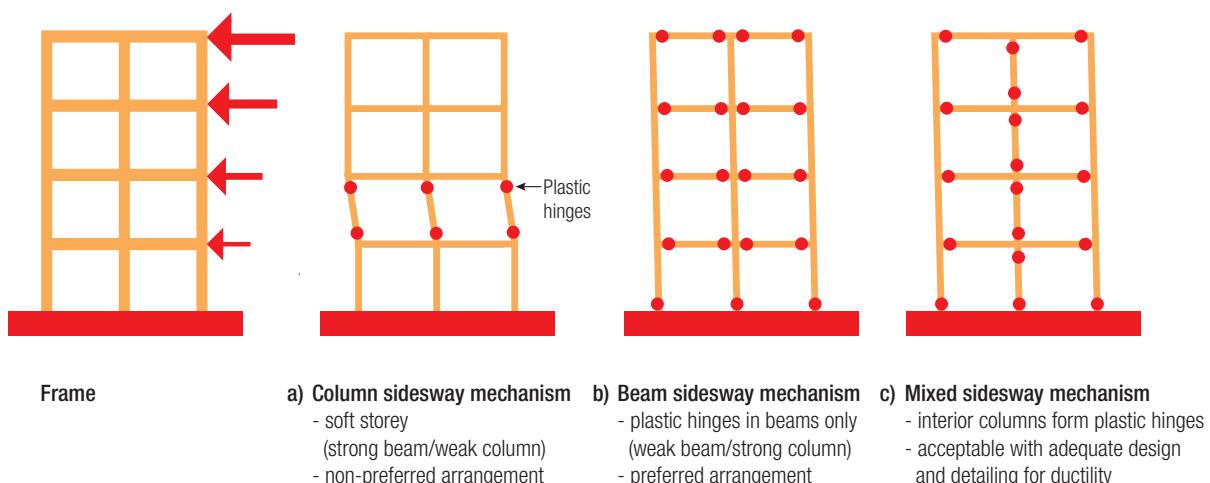


Figure 19 – Column, beam and mixed sidesway mechanisms
(After Goldsworthy³⁵)

5.7.2 Shear walls or structural walls

Walls used for resisting horizontal actions in the plane of the wall (in-plane bending and shear) are generally known as *shear walls* (more correctly known as *structural walls*), although shear is seldom the critical design case. They are very efficient in resisting horizontal actions and usually provide lateral strength more economically than a framed structure using flexure in the columns and beams/slabs. There are many types of shear walls (**Figure 20**) including low-rise squat walls, walls with a large number of openings, tall slender walls, and coupled shear walls. They will act differently depending on their geometry and the lateral actions from the earthquake.

The desired response of a shear wall under lateral earthquake actions is a ductile flexural response although that may not always be possible for squat walls **Figure 20 (a)**. In such cases, a reasonable overload capacity is preferred. Tall shear walls, that act as vertical cantilever elements in buildings, can resist large lateral actions from wind and earthquake while supporting vertical actions. They are very efficient in resisting horizontal actions and usually provide lateral strength more economically than a framed structure using flexure in the columns and beams/slabs. Tall shear walls with good detailing have performed very well during past earthquakes **Figure 20 (c)**.

The behaviour of the wall depends largely on their slenderness. For tall slender walls, the design should aim to achieve ductile flexural yielding at the base of the wall, and shear failure should be avoided. However, for short, squat walls, shear yielding may be acceptable but with an overload capacity.

Most buildings will have stair and lift cores, which will act as the shear walls. These walls together with other walls must be considered in the lateral-force-resisting system for earthquake actions unless specifically designed and detailed not to attract lateral actions. This

requirement is set out in Clause 5.2.3 of AS 1170.4. Shear walls should preferably be arranged so that they do not result in an irregular building in plan, ie it is desirable that the centre of mass and the centre of stiffness should coincide as closely as possible. Shear walls located near the perimeter of the building in a regular manner improve the torsional behaviour of the structure due to earthquakes. However, this may also reduce the axial load on the wall, so sometimes compromises are required to achieve the best layout and efficiency.

Shear walls should preferably continue for the full height of the building and be of the same dimensions (ie not reduce in thickness or length) to avoid vertical irregularity **Figure 21**. In particular, the use of transfer beams to support shear walls above a lower level results in a weak storey below (often the ground floor). Abrupt changes in vertical stiffness concentrate displacement and ductility demands on the floor below, which can often lead to soft storey behaviour and failure. Failure of such arrangements under seismic loads has frequently been observed. Where it is impossible to avoid transfer beams, then the transfer beam and columns should be detailed to provide an appropriate level of earthquake resistance, and an increase in robustness with a reasonable degree of overstrength under seismic actions. In areas of high seismic risk, transfer structures are often designed to remain elastic during the design event whilst surrounding structures are allowed to yield and deform inelastically.

The analysis of walls depends on their height-to-width ratios (h/w). Taller shear walls will act as cantilevers from the base of the building. The design is usually critical at the bottom of the wall, and their design must avoid shear failure.

For walls with $h/w \leq 1.5$ strut-and-tie design should be used, while for $h/w \geq 3.0$ flexural design should be used. In between these values, engineering judgement will be needed to decide whether to use the former and/or the latter design method.

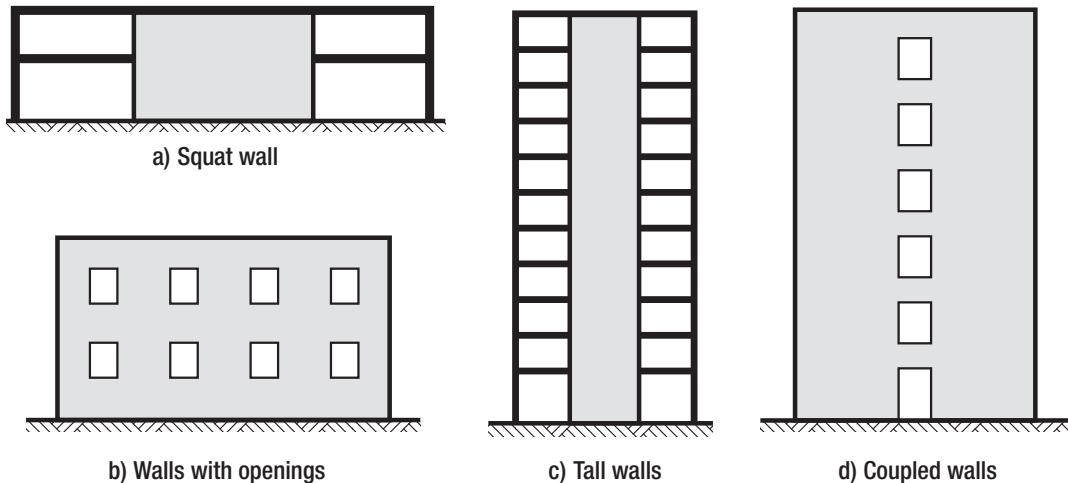


Figure 20 – Some examples of different types of shear walls

Tall walls will usually require large footings with piles, ground anchors or similar to resist the overturning moments **Figure 22**. Alternatively a push-pull system between the ground and basement floors can be used depending on the structural arrangements chosen by the structural engineer **Figure 23**. A push-pull system however, will result in large diaphragm forces in the floor slabs and large shear forces between the basement and the ground floor from the overturning moment.

Consideration must also be given to possible future excavation for a building on an adjoining site where the soil pressures become unbalanced, and the earthquake resistance moves to the bottom of the basement.

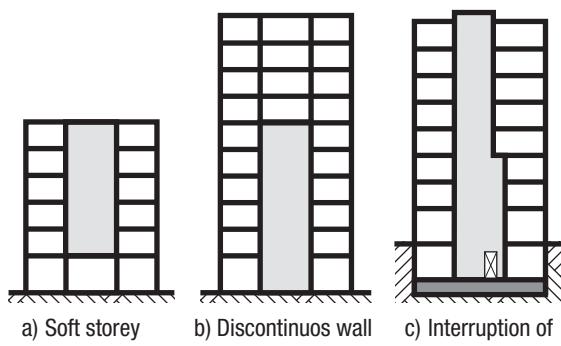


Figure 21 – Some examples of vertical irregularity to be avoided in shear walls

For walls with openings, it may be difficult to achieve elastic behaviour, and careful consideration is needed to ensure responses are essentially elastic even in an extreme earthquake event.

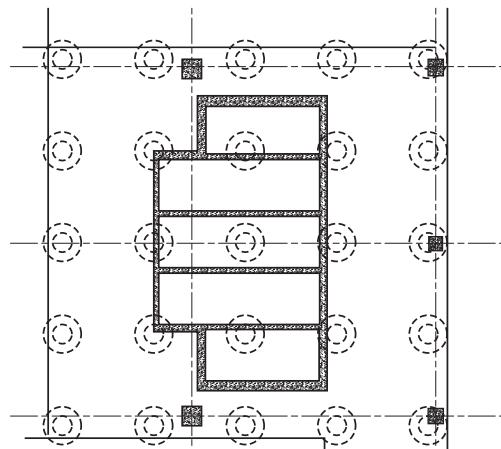


Figure 22 – Example of piled footing to shear walls

As stated earlier, moment-resisting frames deflect in a different manner to shear walls. Interaction between the moment-resisting frame and shear walls arising from the difference in deflection behaviour, will often result in large internal forces within the structure, which need to be considered in the design of and the anchorage of the floor diaphragms (*Refer Section 5.10 Diaphragms*). Also, coupled shear walls **Figure 23(c)** will result in significant bending and shear forces in the header or coupling beams between the adjacent walls. Yielding of the coupling beams usually together with the yielding of the reinforcement at the base of the walls will occur. This is similar to the frame sidesway mechanism shown in **Figure 19 (b)**, ie, the stiff column and flexible beam approach.

AS 1170.4 Table 6.5(A) allows four types of shear walls, viz:

- Ductile coupled walls (fully ductile) $\mu = 4.0$ (outside the Standard)
- Ductile partially coupled walls $\mu = 4.0$ (outside the Standard)
- Ductile shear walls $\mu = 3.0$
- Limited ductile shear walls $\mu = 2.0$.

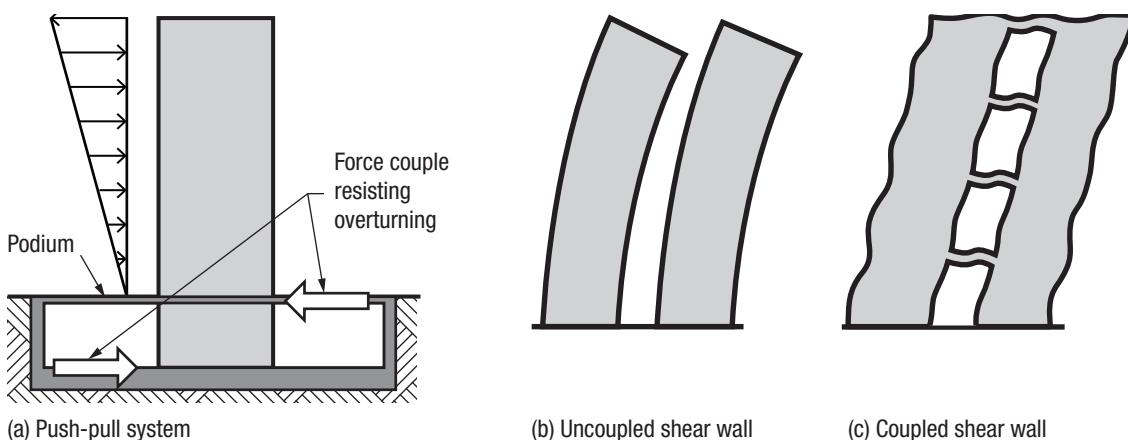


Figure 23 – Cantilever shear walls and uncoupled and coupled shear walls

The first two types of walls lie outside the scope of AS 1170.4 as $\mu > 3$; designers should consult NZS 1170.5³⁴ and NZS 3101³⁸ or alternative documents such as ACI 318M-14¹⁸ or NEHRP Seismic Design Technical Brief Number 6³⁹, as permitted by the BCA.

AS 1170.4 assigns a lower structural ductility factor, μ , to limited ductile concrete shear wall systems which mean they are required to be designed for higher earthquake actions than ductile shear walls. Therefore, limited ductile concrete shear walls often are heavily reinforced in order to develop sufficient strength to justify lower levels of ductility. Clause C1 of AS 3600 allows limited ductile concrete shear walls to be designed and detailed in accordance with the main body of the Standard without further consideration.

Where possible, ductile shear walls should be used because of their improved performance under seismic actions, with reduced design actions leading to smaller wall elements and footings. Clause C5 of AS 3600 sets out the detailing requirements for ductile shear walls.

Loadbearing concrete walls 150 mm thick with a single layer of reinforcement supporting a number of storeys while complying with AS 3600 may not provide acceptable robustness under seismic actions. They are likely to be non-ductile, especially for seismic forces perpendicular to the wall because of drift. It is important not to separate gravity and lateral actions from earthquake actions in the analysis of such walls.

Such walls should have some over strength for earthquake actions and designers need to satisfy themselves that the tension reinforcement in the wall does not rupture, possibly even before the displacement demand corresponding to the return period earthquake level is reached. The reason for this is that highly localised strains may develop at a single crack at the base in these lightly reinforced walls when subject to earthquake ground motions. The failure of the reinforced concrete walls in the Gallery Apartments in Christchurch⁴⁰ is an example.

Analytical studies that show that concentration of plastic deformation in a single crack at the base of the wall is likely to occur for reinforcement ratios up to 0.7%, which is significantly greater than the minimum required value of 0.15% specified in AS3600 and reflects the general lack of ductility in these light centrally reinforced walls.

The possibility of lightly reinforced relatively thin walls buckling in compression after yielding in tension must also be considered.

5.7.3 Boundary Elements

Boundary elements provide the ductility to the edges of the wall in an extreme event by preventing out of plane buckling and improving the confinement of concrete under high compression.

For ductile shear walls, Section C5 of AS 3600 requires

that boundary elements be provided at discontinuous edges and around openings if the vertical reinforcement is not laterally restrained and the calculated extreme fibre compressive stress in the wall $> 0.15f'_c$ **Figure 24**.

Therefore, it could be assumed that if the vertical reinforcement is laterally restrained (which requires two layers of vertical reinforcement, with one layer in each face together with horizontal fitments) and the calculated stress $\leq 0.15f'_c$, then boundary elements are not required.

For low-rise structures not more than four storeys above the structural base and where boundary elements are required, they can be either an integral cast column or additional edge reinforcement of either 2N16 or 4N12 bars which are deemed to satisfy this requirement.

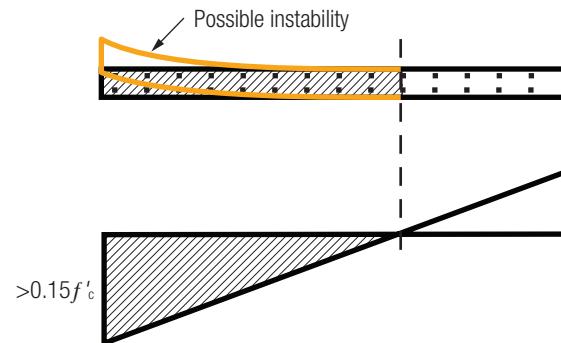


Figure 24 – Potential instability at edges and openings

AS 3600 does not provide any specific guidance on the design of boundary elements of ductile shear walls for structures greater than four storeys in height above their structural base, other than to state that they are to be designed and detailed to resist specified axial forces. It is therefore assumed that boundary elements are designed as columns for the axial and shear actions including seismic actions.

The Standard does not make it entirely clear whether or not nominal boundary reinforcement requires lateral restraint, although the Commentary shows restraint is required. However, Clause C5.3 of AS 3600 requires that reinforcement in a boundary element that is restrained by fitments must comply with Clause 10.7.4 of AS 3600, when the calculated stress is $> 0.15f'_c$ or both Clauses 10.7.3 and 10.7.4 when the calculated extreme fibre stress $> 0.2f'_c$.

Where fitments are required, the minimum thickness of the wall needs to be at least 225 - 250 mm to allow for fixing of the reinforcement. AS 3600 suggests that the requirements for boundary elements may not necessarily result in an increase in their thickness, only that the areas concerned are designed and detailed to resist specified axial forces. **Figures 25, 26 and 27** show examples of boundary elements.

Specific requirements include:

- A reinforcement ratio both vertically and horizontally of not less than 0.0025 is to be provided.
- Where boundary elements are required, the horizontal cross-section of the wall should be treated as an I-beam, where the boundary elements are the flanges and the section of the wall between them is the web.
- Restraint of the longitudinal reinforcement in the boundary elements shall comply with Clause 10.7.4 of AS 3600 or, if the extreme fibre compressive stress calculated as above exceeds $0.2 f'_c$, in accordance with paragraph C4.4 of AS 3600, ie as a column in an intermediate moment-resisting frame with fitments at close centres.

If the wall is greater than 200 mm thick or the design horizontal shear force on the cross-section is greater than $A_g f'_c / 6$, then the reinforcement must be divided equally between the two wall faces, but practicalities are that two layers of reinforcement will be required, as lateral restraint of vertical bars cannot be provided with one layer of reinforcement.

Shear and loadbearing walls if possible should have two

layers of reinforcement, one in each face. Class L mesh reinforcement should not be used for ductile shear walls because of the ductility demands of such walls. Walls also need to be designed for the expected inter-storey sway (drift), perpendicular to the face of the wall.

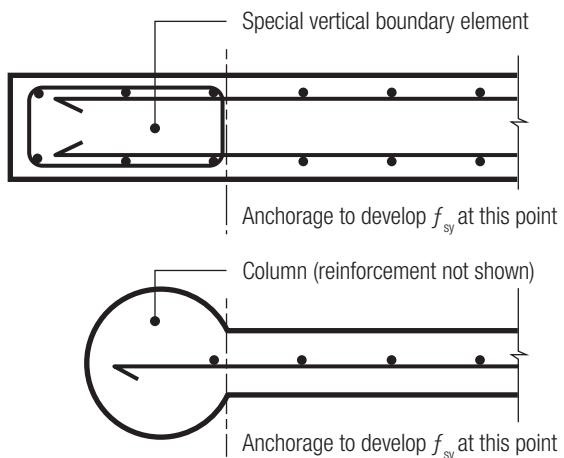


Figure 25 – Typical boundary elements

(Based on AS 3600)

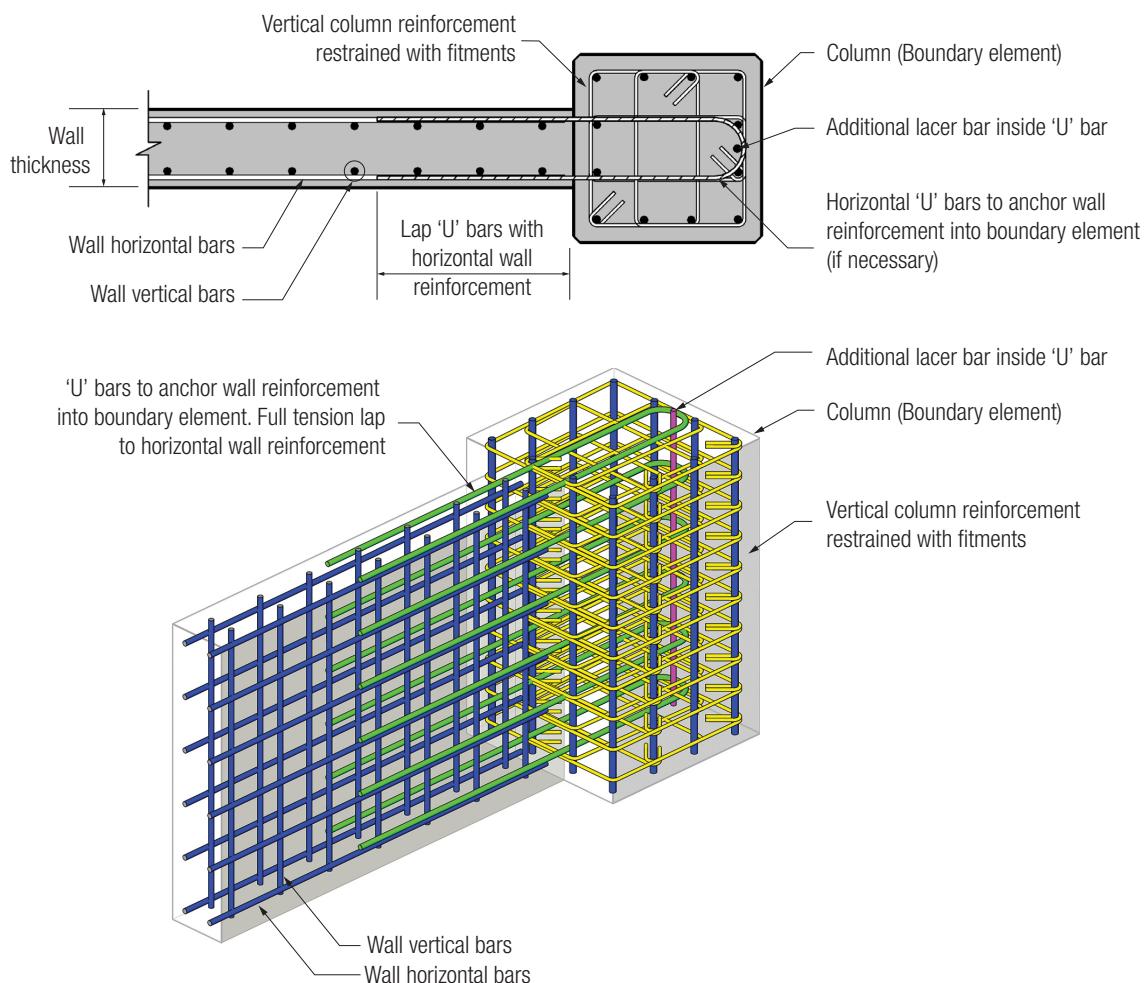


Figure 26 – Example of shear wall with column boundary element

(Detail courtesy of Wallbridge & Gilbert)

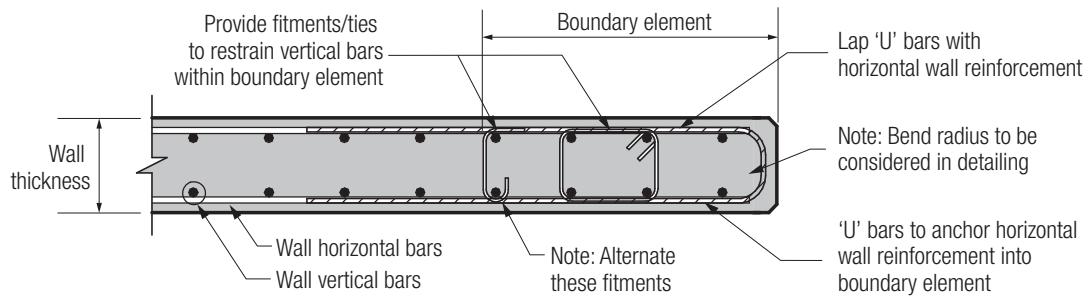


Figure 27 – Example of a shear wall or structural wall with a boundary element

(Detail courtesy of Wallbridge & Gillbert)

An example of the importance of wall boundary elements is shear wall D5-6 of the Hotel Grand Chancellor which failed during the Christchurch earthquake in February 2011 **Figure 28**. While there were many contributing factors to the failure including induced axial loads, lack of flexural yielding due to high axial loads and exceedance of the recommended slenderness ratio, a Report on the Structural Performance of the Hotel Grand Chancellor in the Earthquake of 22 February 2011 by Dunning Thornton Consultants Ltd, concluded that:

“Of the factors listed above that contributed to the brittle failure of the wall, it is the lack of effective confinement that is considered to be the critical factor.

Figure 29 indicates the confinement reinforcement provided in the wall, compared to that required by NZS 3101:1982 Concrete Structures Standard, Part 1: The Design of Concrete Structures, which was the design Standard at the time.

The February earthquake was assessed as being similar to a ‘code’ event having a return period of 1:500 years. However, Dunning Thornton Consultants stated in their report that:

“the recorded vertical accelerations were large and the response of the structure to these may have exceeded code expectations.

The above report by Dunning Thornton Consultants Ltd also stated that:

“If the base of the wall had been more rigorously confined there is a reasonable likelihood that the wall would have survived without failure. If the shaking had been of longer duration then even a confined wall may have failed because loss of the cover concrete would have left the wall quite slender and vulnerable to buckling.



Figure 28 – Brittle failure at the base of shear wall D5-6 of the Hotel Grand Chancellor

(Photograph courtesy Dunning Thornton Consultants Ltd, NZ)

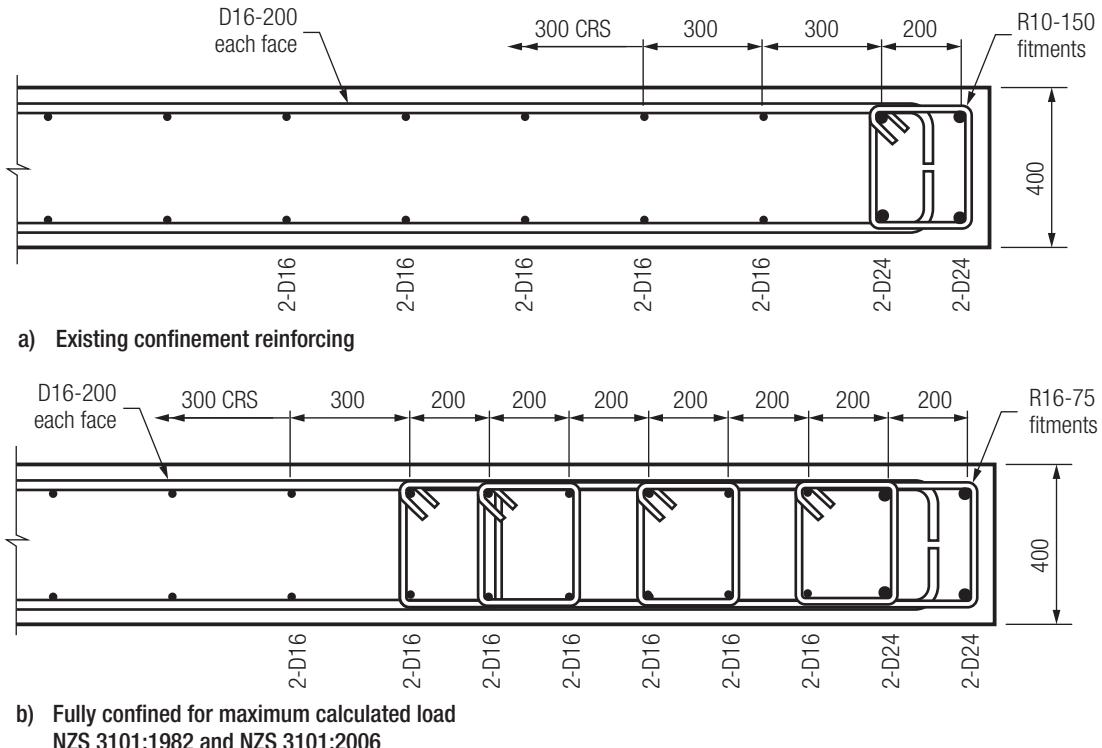


Figure 29 – Confinement requirements for shear wall D5-6 of the Hotel Grand Chancellor

(From Dunning Thornton Consultants Ltd, NZ)

5.7.4 Coupling beams or header beams

Coupling or header beams are used to join shear walls with openings such as stairs or lifts shafts. They are subject to moments and shears under lateral actions in the plane of the wall. They must be ductile and because of the thickness of the walls and the layers of reinforcing, the coupling beam will be at least 250 to 350 mm thick to allow all the reinforcement to be fixed and the concrete placed. Floor slabs alone are ineffective as coupling elements and must not be used. Depending on the relative dimensions of the beam, it can be designed by either flexure or strut-and-tie methods.

AS 3600 does not provide any guidance on the design of coupling beams. Other texts⁹, including a paper by Gurley⁴⁷, provide information on this topic

Figure 30. The beam bars including cogs should be anchored within the confined concrete within the wall reinforcement so that should spalling of the cover concrete occur under seismic actions, the bars can still continue to develop their strength without buckling.

With these beams, detailing of the reinforcement is critical to ensure that the reinforcement is adequately anchored within the body of the concrete and away from any spalled areas, correctly restrained and provides the ductility required. Designers need to remember that as hinges form in these beams, longitudinal reinforcement will cycle between yield in both tension and compression. Under such conditions, good confinement and anchorage are critical.

5.8 Combined systems

Combined systems consisting of shear walls and moment-resisting frames are often used as a structural system.

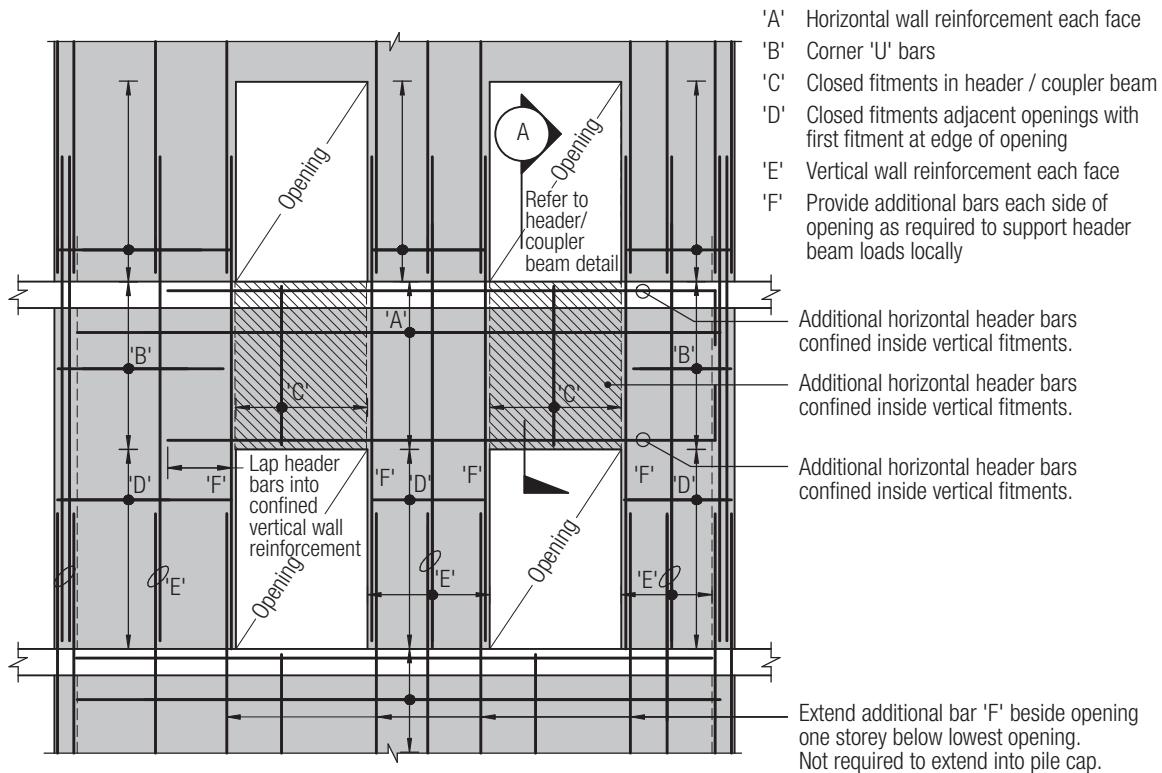
Because of their relative stiffnesses, the shear walls will take most of the lateral actions. However, the relative displacements of the building elements need to be carefully considered, and some actions will be introduced into the moment-resisting frame.

If the moment-resisting frame structure becomes uncoupled from the walls as occurred in the CTV building in Christchurch, NZ, then the moment-resisting frame will take all of the actions, and robust detailing is recommended if possible, to provide a redundant support system. This is where engineering experience and design judgement is needed.

There is currently an inconsistency between AS 1170.4 and AS 3600 as the latter allows a combined system consisting of intermediate moment-resisting frames and ductile shear walls designed in accordance with the Standard and Appendix C, but such systems are not mentioned in AS 1170.4. These systems are more ductile than limited ductile shear wall systems, and they are assigned a correspondingly higher μ value.

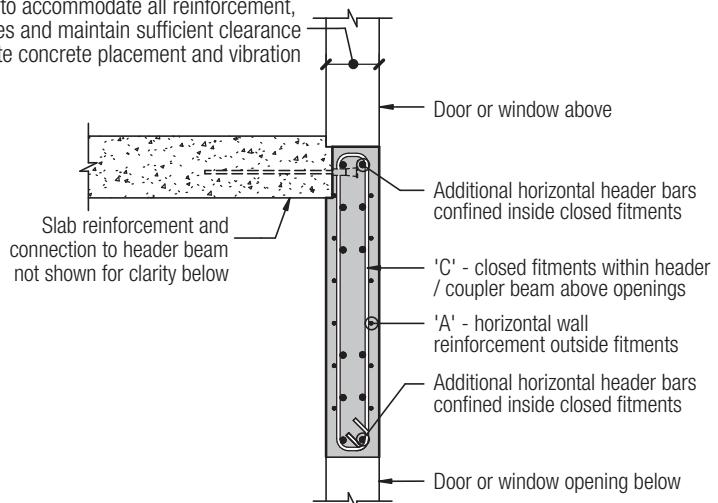
AS 1170.4 allows ordinary moment-resisting frames in combination with limited ductile shear walls with a lower μ value resulting in higher design forces.

All reinforcement terminating in footings, columns, slabs and beams must be adequately anchored to develop yield stress at the junction of the wall and the terminating member.



Typical header/coupler beam elevation

Wall thickness to accommodate all reinforcement, cover, tolerances and maintain sufficient clearance for adequate concrete placement and vibration



Typical header/coupler beam detail

Figure 30 – Elevation and section of a coupling beam
(Detail courtesy of Wallbridge & Gillbert)

5.9 Other concrete structures not covered in Standards

Neither AS 1170.4 nor AS 3600 provides any information on other types of structures, but it can be assumed that they may include such elements as diagonally braced structures, which are unlikely to be used in reinforced concrete buildings.

Detailing should be applied as per other elements (eg, moment-resisting frames and shear walls) as appropriate.

5.10 Diaphragms

In seismic design, diaphragms are the essentially horizontal concrete floor and roof slabs and can include slabs on ground and which tie the structure together. They are a critical but often overlooked element in the design of any building for seismic actions and must be considered early in the design process.

AS 1170.4 makes passing reference to the deflection in the plane of diaphragms in Clause 5.2.5, while AS 3600 in Clause 6.9.4 states that insitu concrete can be assumed to act as horizontal diaphragms. Unfortunately, there is no guidance in either Standard on determining the design actions, the design of diaphragms or the transfer of actions from diaphragms into the vertical elements.

The roles of diaphragms in a building include:

- To carry gravity actions and imposed vertical actions into the vertical loadbearing elements which in turn carry the loads to the footings.
- To provide lateral support to vertical loadbearing elements. If the diaphragm is detached from the vertical loadbearing elements, which are usually heavily loaded, they will become excessively slender, and will almost certainly fail, leading to collapse.

- To transfer the lateral earthquake actions applied at each floor level into the vertical elements.
- To resist out-of-plane forces.
- At lower levels of the building where the floors are at or below the ground level, to resist the lateral shear forces from horizontal actions into the adjacent ground, usually via the retaining walls and the like.

They also have a number of other functions such as redistribution of actions around openings, redistribution of forces due to torsion, and for resisting actions from inclined or offset columns.

One design method has been to consider diaphragms as a deep horizontal beam where the flanges take the tension and compression as required as shown in **Figure 31**. A strut-and-tie approach can also be used and has become the preferred method overseas. Diaphragms can also be rigid or semi-flexible, can be regular or irregular and have large penetrations in them, all of which can complicate their design.

Evaluating all the situations for the detailing of floor diaphragms requires experience and engineering judgement. For example, a building long and narrow in plan may be more flexible than thought, and the deformations may not be able to be accommodated by the walls at either end, resulting in separation of the walls from the diaphragm and possible failure.

The edges of a diaphragm are often an edge beam, which needs to be continuously reinforced with the longitudinal bars fully lapped for tension and compression, restrained for compression and adequately anchored into the concrete walls and columns.

Designers need to study how the forces from the diaphragm distribute to the vertical elements, particularly shear walls. Above all, a good understanding of how these forces are transferred and proper detailing is required.

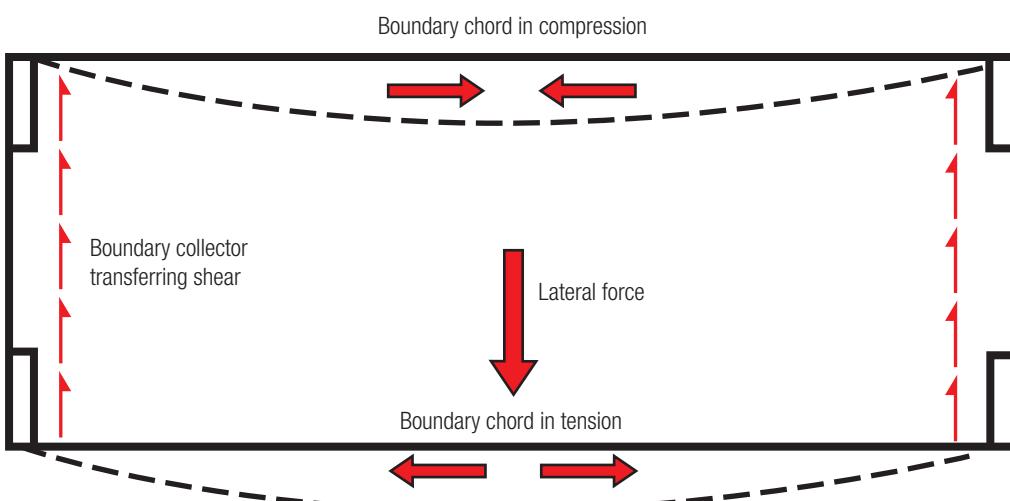


Figure 31 – Floor as diaphragm

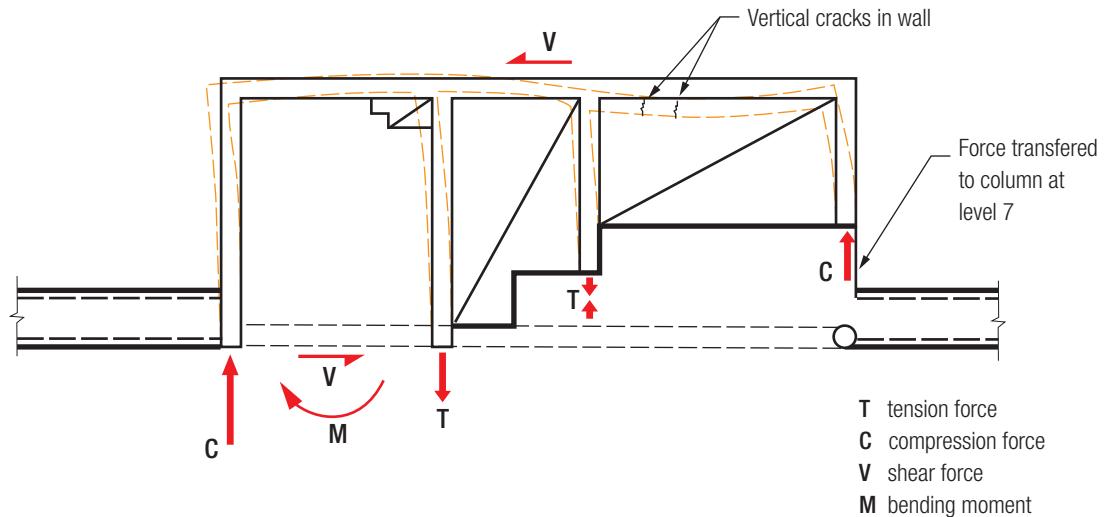


Figure 32 – CTV building and floor as diaphragm

(From the Royal Commission Report)

Volume changes due to creep, shrinkage, thermal and post-tensioning also need to be considered with diaphragms. Where floors are temporally uncoupled from shear walls such as cores and lift shafts to allow for initial shrinkage, axial shortening, and post-tensioning effects, then correct detailing is required to ensure that they will act as one in the final condition and that they are properly connected to the vertical elements.

Diaphragms have a number of components depending on the design model adopted. The boundaries of the diaphragms are either chords in tension and compression or *collectors* because they collect the shear forces and transmit them to the columns and walls. The earthquake forces (which can be significant) must be transferred into the vertical elements from the diaphragm. The

reinforcement used to transfer these forces is known as *drag bars* or *collector bars*.

Many failures of diaphragms in the recent high magnitude Canterbury Earthquakes were observed which highlights the need for a more rigorous approach for the design of these elements.

For stair shafts and lift shafts, where there are openings in the slab for stairs, lifts, and services ducts, designing the connection between walls and floors needs careful attention as there may be only a limited area of wall available to transfer the connection forces. The failure of the CTV building in Christchurch is attributed to the poor connection of floors to the walls as shown in Figure 32 and evident in Figure 6.

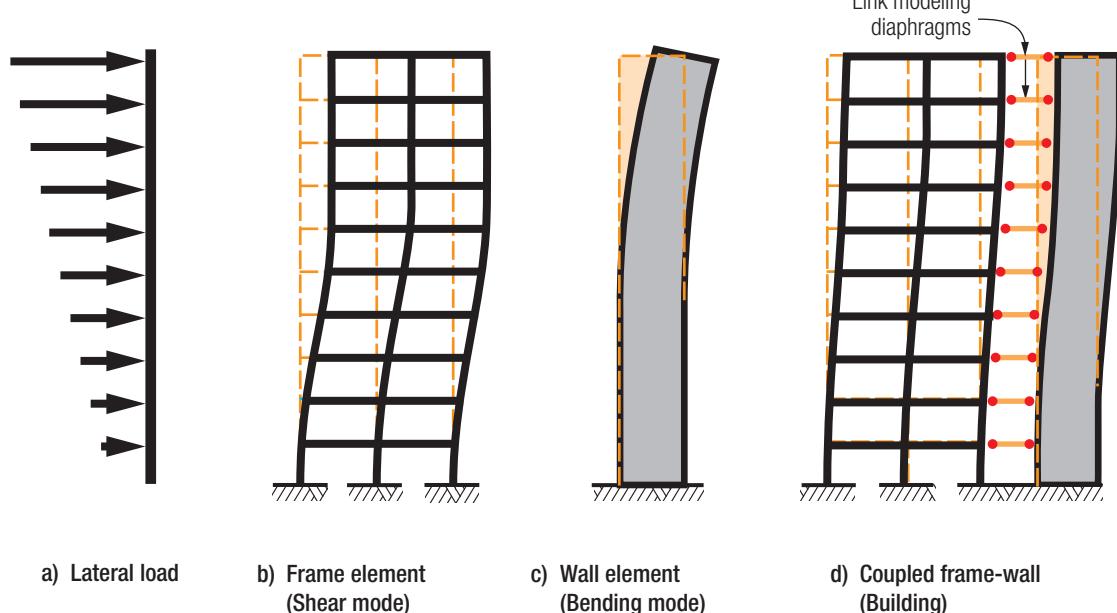


Figure 33 – Deformation patterns for frame and wall elements

(After Gardner, Bull and Carr, 2008⁴²)

Professor Des Bull⁴² and others have cautioned designers that the internal forces within concrete floor diaphragms are considerably more complex than assumed by some of the simplistic design methods used. Moment-resisting frames and shear walls load one another due to different natural modes of deformation. When coupled together by the floor diaphragm they will often result in large internal forces in the diaphragm. There are two main types of forces in diaphragms, viz:

- Inertial forces from the acceleration of the floor
- Transfer forces that result from incompatible deformation between different parts of the structure and, in particular, shear walls and moment-resisting frames.

Different lateral-force-resisting systems within the structure dictate which of these forces, inertia or transfer will dominate as shown in **Figure 33**.

There are a number of specialist texts and technical papers⁴³ that the designer can refer to for the design of diaphragms.

5.11 Floor slabs

5.11.1 General

In-situ floor and roof slabs spanning in either one or both directions and acting monolithically with the supporting beams are capable of acting as a diaphragm with appropriate design and detailing. However where there are a number of excessively large openings, or they are irregular in plan or long and slender to resist earthquake actions other factors must be considered.

(Refer Section 5.10)

5.11.2 One-way slabs

One-way slabs as part of an OMRF are detailed in accordance with the main body of AS 3600, but it is recommended that some continuity of top and bottom reinforcement be provided over the columns and walls.

One-way slabs for IMRFs are detailed in accordance with the requirements for slab reinforcement in Appendix C of AS 3600, which is similar for beams for an IMRF (eg provision of reinforcement, continuity, anchorage and lapping except that lapped splices located in a region of tension or reversing stress do not need to be confined by closed fitments at the splices).

5.11.3 Flat slabs

General

Flat slabs are two-way slabs, including flat plates, slabs with drop panels, waffle slabs and similar.

AS 3600 is not entirely clear on what detailing is required for flat-slab construction forming part of an OMRF. However, the designer could reasonably assume that it should be detailed in accordance with the main body of the Standard, with no specific detailing required. However, additional bottom reinforcement should be provided at the columns as for IMRF's in the column strip or drop panel to avoid failure. **Figures 34, 36 and 37.**



Figure 34 – Failure of slabs with no bottom reinforcement, Newcastle Workers Club

(Photo courtesy Cultural Collections, the University of Newcastle, Australia)

For flat-slab construction forming part of an IMRF, there are additional requirements to ensure that ductility and continuity conditions are met at the column and middle strips along the line of supports.

Appendix C of AS 3600 sets out the following criteria:

- All reinforcement resisting the portion of the slab moment transferred to the support is to be placed within the column strip.
- A proportion of this reinforcement is to be evenly distributed in a narrower zone measuring 1.5 times the thickness of the slab or drop panel either side of the column, or capital.

This proportion is the greater of 0.5 or

$$\frac{1}{1 + 2/3\sqrt{[(b_1 + d_o)/(b_t + d_o)]}}$$

where:

b_1 = the size of rectangular (or equivalent rectangular) column, capital or bracket, measured in the direction of the span for which moments are being determined.

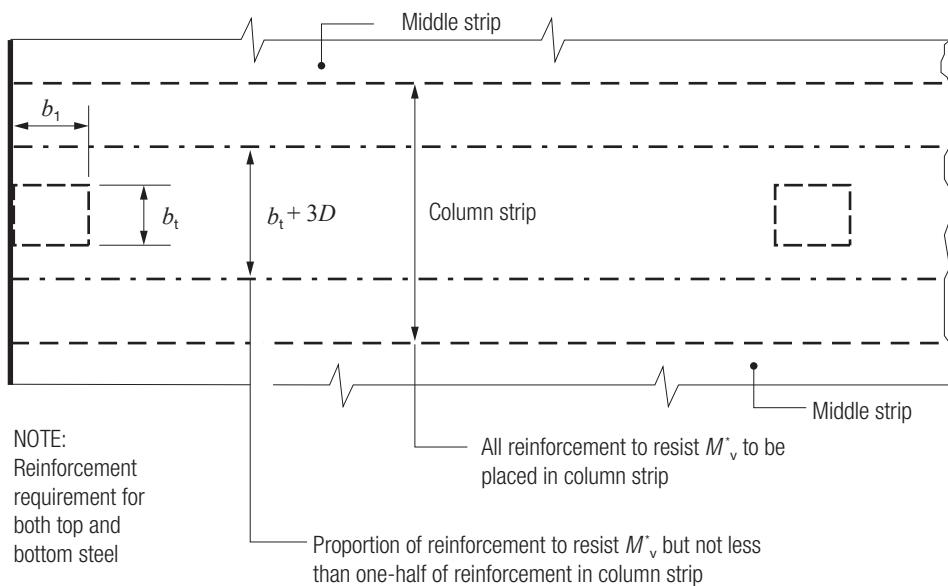
b_t = the size of rectangular (or equivalent rectangular) column, capital or bracket, measured transversely to the direction of the span for which moments are being determined.

d_o = distance from the extreme compressive fibre of the concrete to the centroid of the outermost layer of tensile reinforcement.

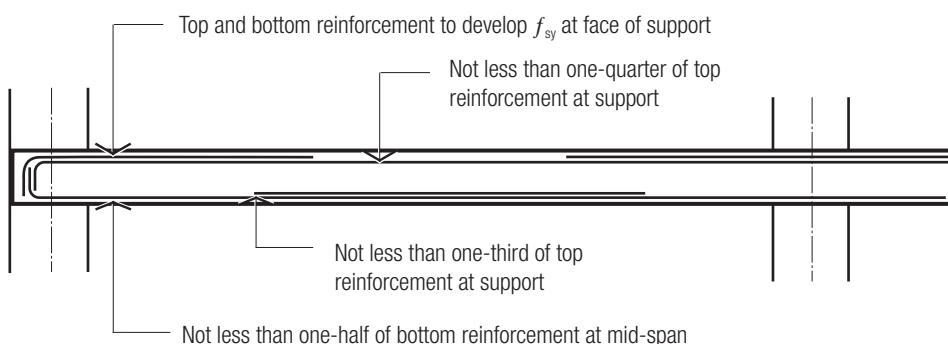
- At least 25% of the top reinforcement at the support in the column strip is run continuously through the span.
- At least 33% of the area of top reinforcement at the support in the column strip is provided in the bottom of the column strip, again running continuously through the span.

- At least 50% of all bottom reinforcement at mid-span is to be continuous through the support such that its full yield strength is developed at the face of the support.
- At discontinuous edges of the slab, all top and bottom reinforcement at a support is to be capable of developing its yield strength at the face of the support.

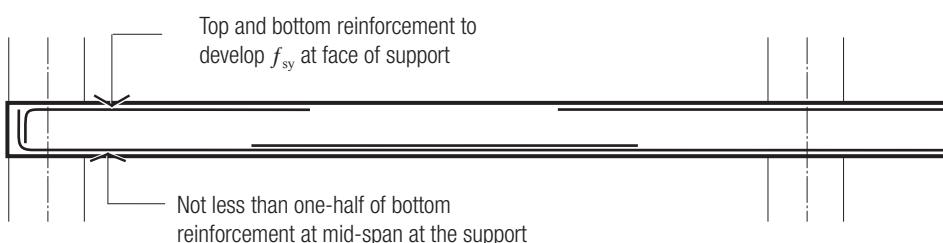
These requirements are illustrated in **Figure 35**.



Plan view at columns



Section of column strip



Section at middle strip

Figure 35 – Reinforcement detailing for flat slabs
(From Clause C4.3.2 of AS 3600)

With flat-slab construction, it is important to ensure that the slab-column connection can withstand the deformation and moments arising from the drift of the primary lateral-force-resisting system without shear failure and subsequent collapse. The most important factor influencing the inelastic deformation that can be sustained in the slab is the level of axial load to be transferred to the column at the joint zone. As the magnitude of axial load increases, so the available ductility decreases. Failure can be brittle in character, and to prevent this type of failure, secondary reinforcement should be placed in the bottom of the slab, band beam or drop panel at the column slab intersection in each direction to resist the gravity actions in a tensile membrane action **Figures 36 and 37**. The addition of a small quantity of bottom face reinforcement running through the column joint provides cheap insurance against collapse following a punching shear failure of the concrete floor plate. ACI 318M-14¹⁸ refers to such reinforcement in the robustness/structural integrity requirements and while not specifically required by AS 3600, is considered to be good practice.

According to ACI 352.1R-11, Guide for Design of Slab-Column Connections in Monolithic Concrete Structures, at interior columns the additional bottom face reinforcement passing within the column core in each principal direction should have an area not less than:

$$A_{sm} = \frac{0.5\omega_u l_1 l_2}{\emptyset f_{sy}}$$

where

ω_u = factored uniformly distributed load, but not less than two times the slab dead load (N/mm^2)

l_1 = length of span in direction that moments are being determined, measured centre to centre of supports (mm)

l_2 = length of span in direction perpendicular to l_1 , measured centre to centre of supports (mm)

If bars can only be placed in one direction, provide $2A_{sm}$ in that direction. For edge connections in the direction perpendicular to the slab edge, provide two thirds A_{sm} and for corner connections, provide $0.5A_{sm}$ in each principal direction. Where A_{sm} varies for adjacent spans, the larger value should be used.

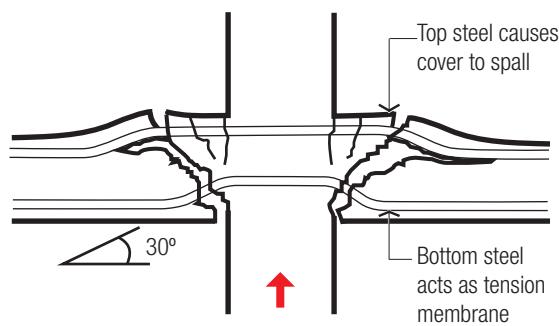


Figure 36 – Tensile membrane steel at column-slab intersection

(From Mochle⁹)

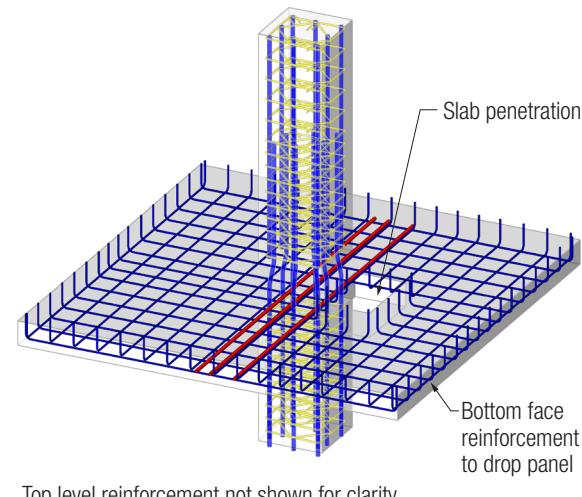


Figure 37 – 3D view of steel at column-slab intersection

(Detail courtesy of Wallbridge & Gilbert)

5.12 Beams

Beams including band beams that are part of an OMRF only need to be designed and detailed in accordance with the main body of the Standard. Again it is recommended that continuity of reinforcement be provided top and bottom, consistent with ACI 318M-14¹⁸, to provide increased ductility and robustness/structural integrity.

Beams in IMRFs must be continuously reinforced to ensure adequate ductility as shown in **Figure 38**.

If yielding occurs, the reinforcement will not remain within the elastic part of the stress-strain curve, and Bauschinger softening will occur under cyclic loading. The 'Bauschinger effect' is the change in the stress-strain relationship that occurs when a reinforcing bar is yielded in tension or compression, and the direction of the stress is reversed. The distinct yield point is lost, and the stress-strain relationship takes on a curvilinear form.

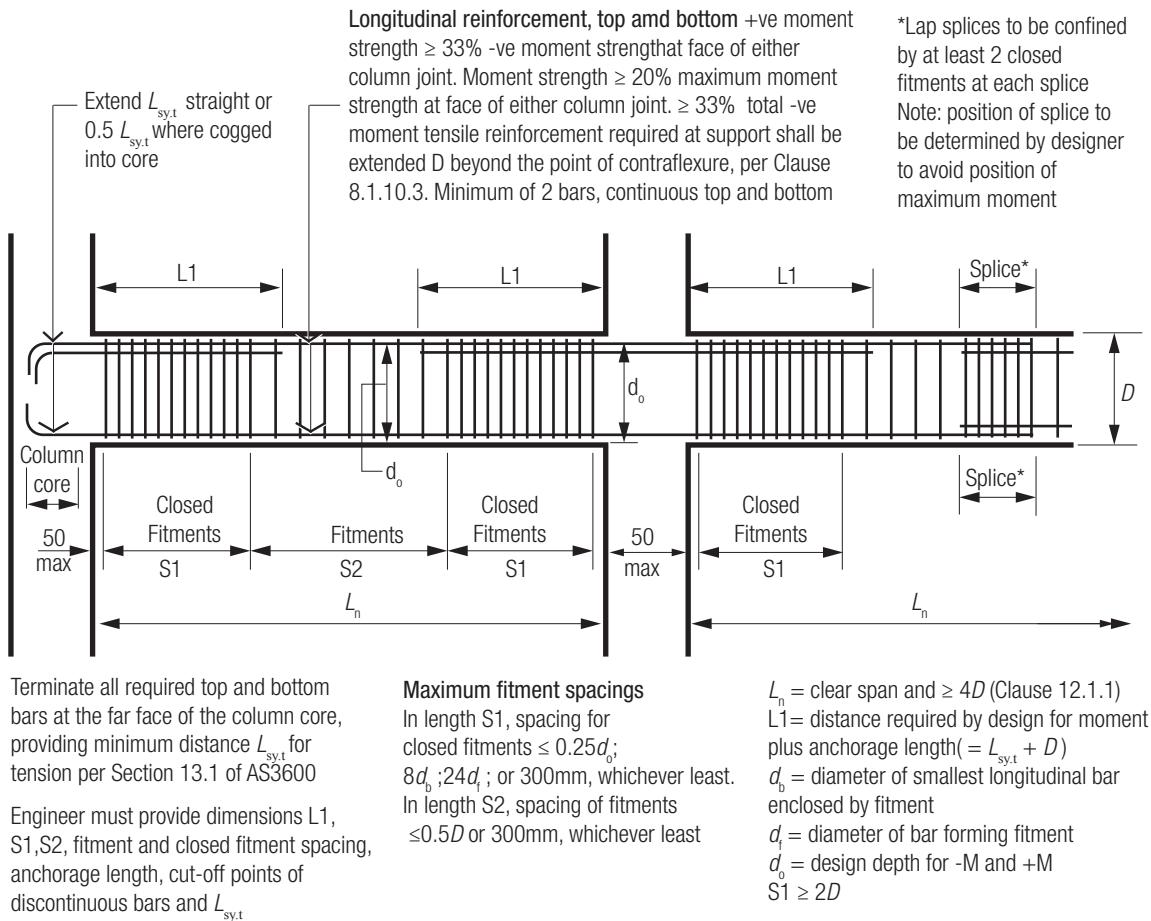


Figure 38 – Typical detailing of the beam reinforcement for IMRF structure
(Based on AS 3600)

The stable hysteretic* (reversed cyclic loading) response of the potential plastic hinge region can be diminished through the 'pinching' of the hysteretic loop due to the influences of shear degradation of the region. This could be as a result of inadequate transverse reinforcement or poor construction joints.

The effect of reversing moments is concentrated at the junctions between beam and column. For an IMRF beam, Appendix C requires the following:

- The top and bottom face of the beam should be continually reinforced.
- The positive moment strength at a support face is to be not less than 33% of the negative moment strength provided at the face of the support; and
- Neither the negative nor the positive moment strength at any section along the member length is to be less than 20% of the maximum moment strength provided at the face of the support.
- All longitudinal reinforcement must be fully anchored beyond the support face so that at the face the full yield strength of the bars can be developed. This requires that:

- longitudinal reinforcement is continuous through intermediate supports and
- longitudinal reinforcement extends to the far face of the confined region of external columns and is fully anchored by a cog or hook or similar.

Lapped splices in longitudinal reinforcement, located in a region of tension or reversing stress, are to be confined by a minimum of two closed fitments at each splice to inhibit the possibility of non-ductile failure at this point.

The position of the maximum moment under seismic load will be dependent on the magnitude and direction of the earthquake. The splice should therefore be located at a position of known moment, preferably in the middle third of the span, unless the designer is confident that the splice is sufficiently confined that it can be safely located elsewhere in the span.

Shear failures of beams tend to be brittle. Maintaining a stable hysteretic response of the plastic hinge regions requires that the compression bars be prevented from buckling. In designing hinge regions, the designer must assume that significant spalling of concrete cover will

*A hysteretic loop is often a plot of force vs deflection or rotation for cyclic loads which under ultimate conditions will usually diminish with successive cycles due to yielding of the reinforcement and crushing of the concrete.

occur and that the compression bars must rely solely on the transverse support provided by fitments. Limitations on the maximum fitment spacing are required to ensure that the effective buckling length of the compression bars is not excessive and that concrete within the fitment has a reasonable confinement **Figure 39**. Furthermore, due to the possible occurrence of the Bauschinger effect and the reduced modulus of elasticity of the steel, a smaller effective length between fitments must be considered for bars subject to flexural compression, rather than compression alone.



Figure 39 – Failure of a beam column joint at Christchurch 2011.

Note how the bottom bars of the beam have not been adequately anchored in the confined region of the column and cannot develop the stress in the bars

(Photograph courtesy Peter McBean)

Appendix C of AS 3600 specifies a minimum area of shear reinforcement and requires the following.

- Shear reinforcement shall be perpendicular to the longitudinal reinforcement throughout the length of a member with at least two legs and a maximum spacing of $0.5D$.
- The area of shear reinforcement $A_{sv} \geq 0.5 b_v s / f_{sy,f}$ ie 42% greater than the minimum shear reinforcement required by Clause 8.2.8 of AS 3600, with closed fitments provided over a minimum distance of $2D$ from the face of the support. The first fitment should be placed no more than 50 mm from the support face, and the remainder spaced at $0.25d_o, 8d_b, 24d_f$ or 300 mm, whichever is least, where:
 - b_v = effective width of the web for shear.
 - s = centre to centre spacing of fitment.
 - $f_{sy,f}$ = yield strength of reinforcement used for fitments.
 - D = overall depth of cross-section in the plane of bending.
 - d_o = the distance from the extreme compression fibre of the concrete to the centroid of the outermost layer of tensile reinforcement, but not less than $0.8D$.
 - d_b = the nominal diameter of the smallest longitudinal bar enclosed by the fitment, and
 - d_f = the diameter of the bar forming the fitment.

One of the difficulties with closed fitments is the fixing of reinforcement, particularly the top reinforcement.

Since tension in vertical fitment legs acts simultaneously to restrict longitudinal bar buckling and to transfer shear force across diagonal cracks, the vertical area of fitment must be sufficient to satisfy both the requirements for bar buckling and those for shear resistance. Note these requirements do not preclude efficient fabrication techniques such as loose bar detailing as recommended in the CIA's Recommended Practice³³.

ACI 318M-14¹⁸ allows fitments in beams to be made up of two pieces with an open fitment having hooks at both ends and closed by a fitment (crosstie) where they are required to be closed (AS 3600 allows a similar arrangement for internal fitments for columns but not for beams). Consecutive fitments (crosstie) engaging the same longitudinal bar shall have their 90° cogs at opposite sides of the flexural member in a similar manner to columns as shown in **Figure 40**. If a slab confines the longitudinal reinforcing bars secured by the fitment (crosstie) on only one side of the flexural frame member, then the 90° cogs of the fitment (crosstie) shall be placed on that side.

Regarding the location of the fitment hooks that anchor the fitments into the confined concrete of the beams (or columns), the usual procedure is to locate these in the compression zone of the member. When a slab is located adjacent to the beam, the stirrup hook should be located on this side as spalling of the cover concrete will not occur. Unfortunately for columns, this is rarely the case. Due to the cyclic nature of earthquakes, moment reversal will occur and compression zones will become tension zones.

The commentary to ACI 318M-14 points out that:

“A longitudinal bar within a stirrup hook limits the width of any flexural cracks, even in a tension zone.

It goes on to state :

“such a stirrup hook cannot fail by splitting parallel to the plane of the hooked bar.

This suggests that the fitment hooks will perform adequately when located in both tension and compression zones.

The minimum length of the straight hook extension for seismic stirrup/ties in ACI 318M-14 ($6d_b$ or ≥ 75 mm) is slightly longer than that required by Clause 13.1.2.7 of AS 3600 Standard hooks and cogs ($4d_b$ or ≥ 70 mm). As Appendix C of AS 3600 contains no additional requirements for the anchorage of fitments, the existing requirements are considered suitable for both ordinary and intermediate moment-resisting frames.

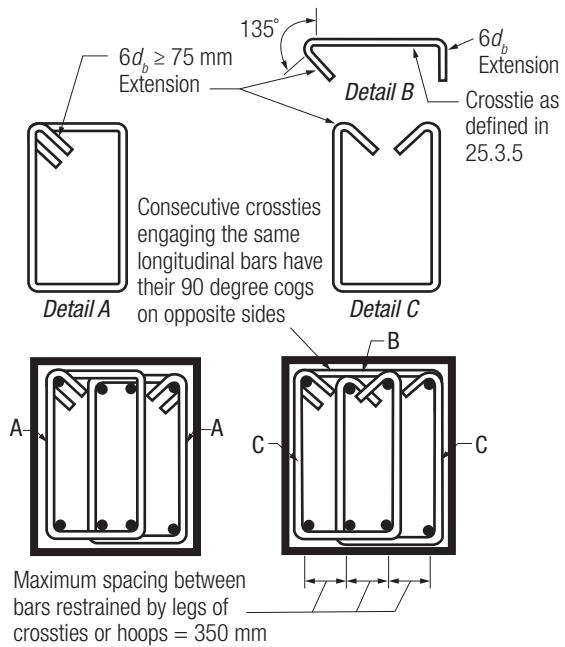


Figure 40 – Typical detailing of beam shear reinforcement
(From AC 318M-14)

Band beams are a very popular form of beam construction in Australia; they are shallow beams and must be detailed in accordance with AS 3600. As previously discussed, a proportion of the bottom face bars should run through the beam column joint for ductility and robustness/structural integrity. (Refer Section 5.11.3)

There can be problems in placing all the required column joint fitments around the column bars within the height of the band beam when designed as part of an IMRF. Irvine and Hutchinson⁴⁴ suggest that in fully ductile frames, the beam column joint fitments would usually be placed with one fitment set directly on top of the next set with no clear space between, and recommend that for frames of limited ductility, the principal beam ratios (A_{st}/bd) be limited to 0.02 or less, so as to reduce the problems of placing the beam column joint fitments. This needs to be considered especially in the band beam external column joint connections to ensure adequate ductility in the column to prevent a plastic hinge and potential collapse mechanism forming in the column. If this proves impractical, the mechanism shown in Figure 19 (c) may be considered, provided rigorous analysis and careful detailing are undertaken. The longitudinal bars should be spaced equally over the band beam width both adjacent to the column and within the column width. A balanced placement of the outer bars each side of the column is important to minimise induced torsion.

Detailing of band beams is as for normal beams.

Band beams should not be used in Special Moment-resisting Frame systems and require careful detailing for an IMRF.

5.13 Transfer structures

Transfer structures are commonly used where columns and walls have to change position in a vertical direction because of a different layout of the vertical structural elements of the structure within the building. Wherever possible they should be avoided but in a real structure this may not always be practicable.

Unfortunately, experience in major earthquakes worldwide has shown that transfer structures are very prone to failure. Careful consideration should, therefore, be given to the design of transfer structures, including their detailing and an assessment of alternative load paths, should they fail. The design actions that could occur should be carefully reviewed together with consideration of the robustness of the transfer structure (Refer to earlier discussion in Section 5.7.1).

A solution could be to design and appropriately detail transfer structures elastically for the full earthquake loading.

5.14 Columns

5.14.1 General

The most important structural elements in a building during earthquakes are the columns. Failure of columns will usually lead to the collapse of the building. Their design and detailing to deliver satisfactory performance is therefore the single most important aspect of seismic design Figure 41.



Figure 41 – Failure of a column at the Hotel Grand Chancellor due to poor confinement

(Photograph courtesy Peter McBean)

The axial stress levels in the columns have a large effect on their seismic behaviour. Columns should be designed with sufficient reserve capacity to accommodate both the design actions and drift associated with the design event⁴⁵. Highly loaded and poorly detailed columns failed in the Christchurch CTV building in an overload situation. Most of the building became separated from the lift core because the anchorage between the core and the rest of the building was inadequate. In a similar manner, columns failed in the Newcastle Workers Club in 1989 because they were heavily loaded and incorrectly designed and detailed.

It is strongly recommended that where possible the axial load should be kept below the balance point shown in **Figure 42**.

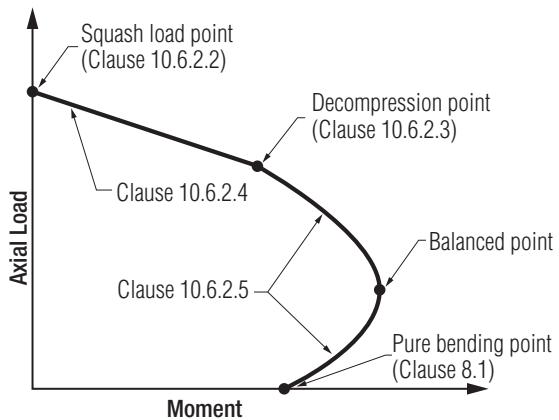


Figure 42 – Axial Load versus Moment Diagram

(From AS 3600)

As discussed in **Section 5.6**, designers should try to ensure that any plastic hinges, which form under seismic actions, occur in the beams rather than the columns. This is achieved by ensuring the flexural capacity of the column is higher than that of the beam by a significant margin to allow for any 'overstrength' due to design or materials. This is known as the '*weak beam/strong column*' philosophy. Although it may not always be possible to achieve this, especially with band beams, care should be taken that a catastrophic collapse, especially one due to brittle shear failure of the column, will not occur.

A rule of thumb is that the total column strength above and below a particular joint should be 25 to 30% greater than the equivalent total beam moment strengths on either side of the column.

In many cases, the ultimate compression strain of unconfined concrete is inadequate to allow the structure to achieve the design level of ductility without excessive spalling of cover concrete. Adequate transverse reinforcement must, therefore be provided to confine the compressed concrete within the core region to maintain its axial-load-carrying capacity and to prevent buckling of the longitudinal compression reinforcement and subsequent failure. Plastic hinge regions are particularly susceptible where substantial axial forces are present, eg in columns where inelastic deformations must occur to develop a full hinge mechanism. Note that this may occur even where the design is based on weak beam/strong column philosophy, such as at the base of all columns **Figure 19(b) and 19(c)**.

Depending on whether the column is part of an ordinary, intermediate or special moment-resisting frame, the spacing of the fitments and splice points will vary. **Figure 43** indicates the requirements for an IMRF based on the requirements in the 2001 version of AS 3600.

Where columns are used in framed stairwells, they typically are reduced in height due to the stair half landings and, therefore are more susceptible to brittle shear failures. A similar problem occurs when deep beams are used externally with short columns.

5.14.2 Confinement

Close-spaced fitments (also referred to as ties, stirrups, ligatures or helical reinforcement) acting in conjunction with longitudinal reinforcement restrains the lateral expansion of the concrete. Fitments enable the concrete to withstand higher levels of compression. Circular or helical fitments, due to their shape, are placed in hoop tension by the expanding concrete and provide confinement to the enclosed concrete **Figure 44 (a)** but they must be correctly anchored. For circular-shaped fitments, they can be spliced by welding or anchored by providing two 135° fitment hooks around adjacent longitudinal bars or bundles. For helical reinforcement, adequate anchorage of the end is provided by one and a extra half turns of the helix.

Rectangular fitments apply full confinement only near their corners as the pressure of the concrete bends the legs outwards. This tendency should be counteracted by the use of internal fitments or interconnected closed fitments at all vertical bars; this has the additional benefit of increasing the number of legs crossing the section. The profiles of the unconfined zones of concrete between longitudinal bars are shallower, and consequently a greater area of concrete is confined. The presence of a number of longitudinal bars, enclosed by closely spaced fitments, will also significantly aid confinement **Figure 44(b), (c), (d) and (e)**.

Also, all fitments must have their ends fixed so that they are enclosed in the concrete with 135° hooks; 90° cogs must not be used except for internal fitments or crossties.

Appendix C of AS 3600 for IMRF's requires the provision of closed fitments over a distance of either:

- the maximum dimension of the column cross-section, D , or
- one-sixth of the least clear distance between consecutive flexural members framing into it ($L_o/6$).

For critical columns in high seismic risk areas, the fitments are often provided at close centres for the whole height of the column. The Northridge earthquake showed that when the fitments were not at close centres, the columns often failed.

Appendix C of AS 3600 for an IMRF refers designers to the Clauses 10.7.3 and 10.7.4 in the Standard for confinement and restraint of longitudinal reinforcement. Where the concrete strength, f'_c , in the columns is ≤ 50 MPa, the spacing of the closed fitment or helix for single bars is to be the least of D_c or $15d_b$, with the first fitment located at 50 mm from the support face. For bundled bars, the spacing of fitments is half those for single bars (Note: D_c is taken as the smaller column dimension). Special rules apply for all columns where f'_c is > 50 MPa as set out in Clause 10.7.3 of AS 3600. Note that Appendix C4.4 of AS 3600 requires that the first tie is located at half the required spacing from the support face. Note that in the 2001 version of AS 3600, the closed ties that extend over the distance D or $L_o/6$ were required to be spaced at maximum centres of $0.25d_o$, $8d_b$, $24d_f$ or 300mm with the first tie located a maximum 50mm from the support face, or $0.5s_c$. This requirement appears to have been lost in the current revision of

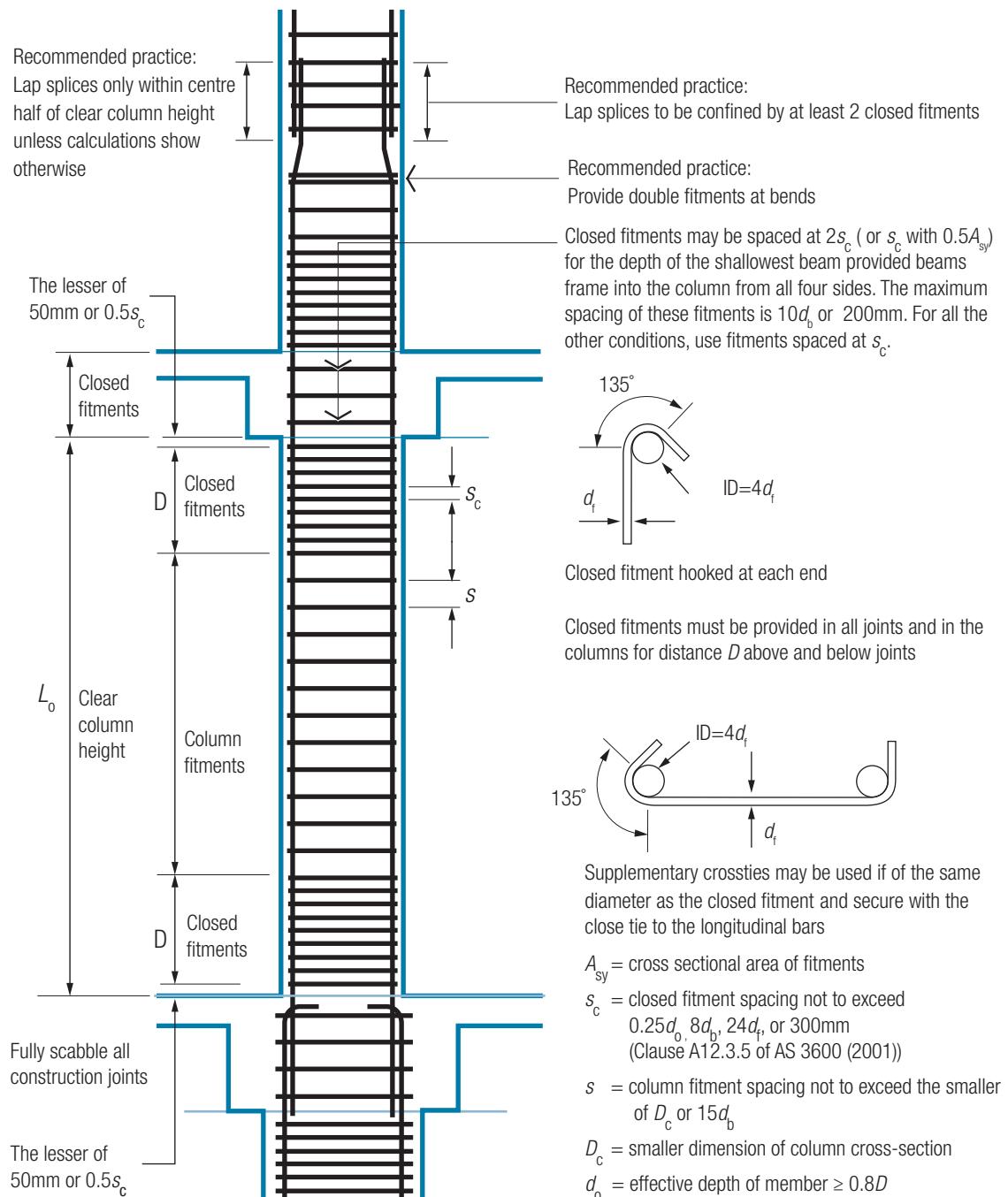


Figure 43 – Detailing for an IMRF column

AS 3600, but is reflected in the detailing in **Figure 43** and is supported by other standards such as NZS 3101.1 and ACI 318M-14.

Some examples of detailing of columns on site are shown in **Figure 45**.

The overall cross-sectional area of the fitment must also be enough to satisfy the shear requirements of the columns under lateral actions – a requirement that has sometimes been overlooked.

Concerns have been raised regarding the ductility and displacement capacity of high-strength concrete columns. Designers should therefore not use high-strength concrete columns were earthquake design is a critical design consideration - particularly for highly stressed columns, for IL4 buildings or for columns that support a transfer structure or similar.

5.14.3 Lapped splices

The splices required in the column reinforcement of multi-storey buildings are typically made on a floor-by-floor basis. For an OMRF, this is usually just above the

floor level. For an IMRF the splice can be just above the floor level if calculations prove satisfactory, otherwise the recommended practice is to provide the splice at the mid-height of the columns. For an SMRF, splices would usually be at the mid-height of the column. The designer must therefore ensure that these are detailed and located such that failure will not occur under earthquake action. Splicing is achieved by the use of overlapping parallel bars. Force transmission occurs due to the bond between the bars and the surrounding concrete, as well as to the response of concrete between adjacent bars. Under severe cyclical loading, column splices tend to progressively 'unzip'. Further, where large forces in the column reinforcement are to be transmitted by bond, splitting of the concrete can occur. To prevent these occurrences, fitments are required to provide a 'clamping force' to the longitudinal reinforcement against the core concrete. In circular columns, the clamping force is provided by helical or circular fitments.

Unless the designer has checked the capacity, splices preferably should not be placed in potential plastic hinge regions. While transverse fitments may ensure

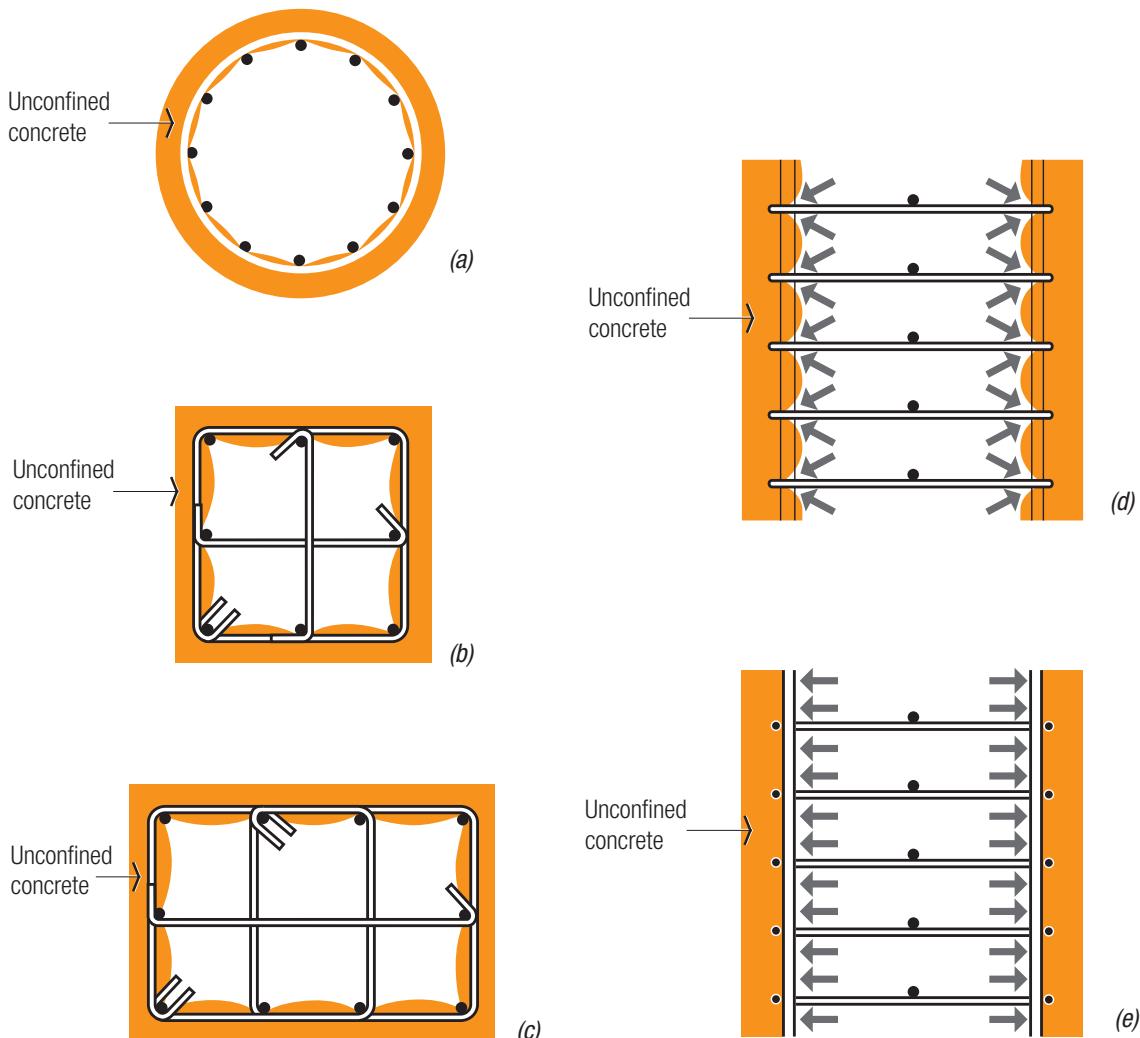


Figure 44 – Confinement of column sections by transverse and longitudinal reinforcement
(From Paulay and Priestley⁶)



(a) Column reinforcing to a building with post-disaster requirements



(b) Column reinforcing for IMRF

Figure 45 – Examples of column reinforcement

(Photographs courtesy John Woodside)

strength development of the splice under cyclical loading at up to, but still below, the yield stress of the reinforcement, they will not ensure a satisfactory ductile response. This is especially true where large-diameter bars are lapped in the plastic hinge zone. The splice will fail after a few cycles of loading large enough to induce inelastic behaviour in the longitudinal reinforcement, with a consequent gradual deterioration of bond transfer between the bars⁶. For example, for all frame types, a plastic hinge will try to form at the base of first-storey columns unless columns are stiffer than the beams. Consideration should be given to carrying the column bars above first-floor level before splicing. A less preferred alternative would be to locate the splice at mid-height of the column.

If the formation of plastic hinges is precluded, then splicing of the longitudinal bars by lapping can be detailed just above the floor level. New Zealand practice allows columns that have greater than 1.25–1.4 times the flexural strength of the adjoining beams, provided the column shear strength is similarly higher (ie matching the column flexural capacity), to be spliced just above the floor. Such columns are unlikely to yield and form plastic hinges.

Splicing by butt welding or by mechanical couplers can be used where bar congestion can be a problem but is not common. Under no circumstances, should these be situated in a potential plastic hinge region, to help

ensure a weak beam/strong column failure.

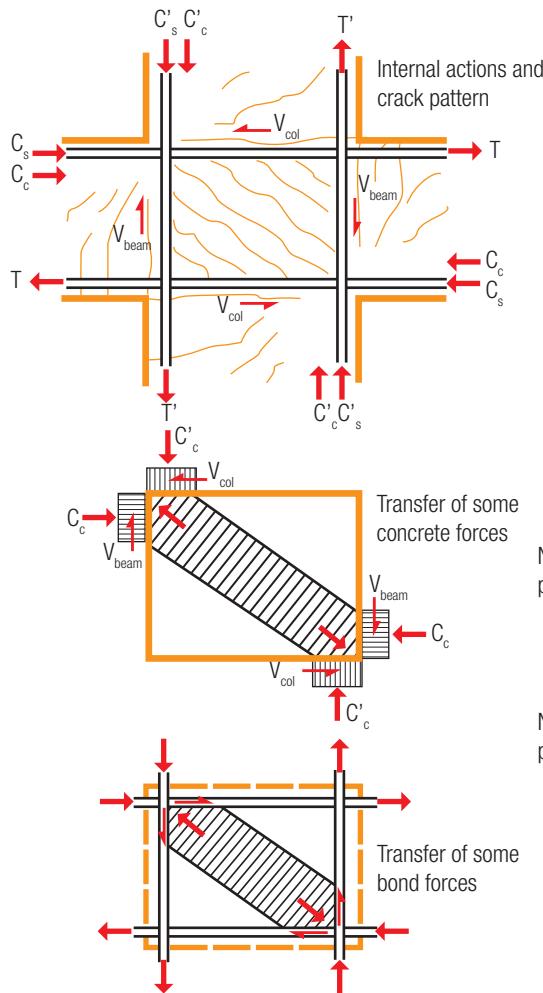
Site welding of bar splices requires special consideration and care during execution. Lap welding should be avoided. Butt welding is acceptable, provided it is carried out using a proper procedure in accordance with AS 1554.3²⁵.

5.14.4 Beam column joints

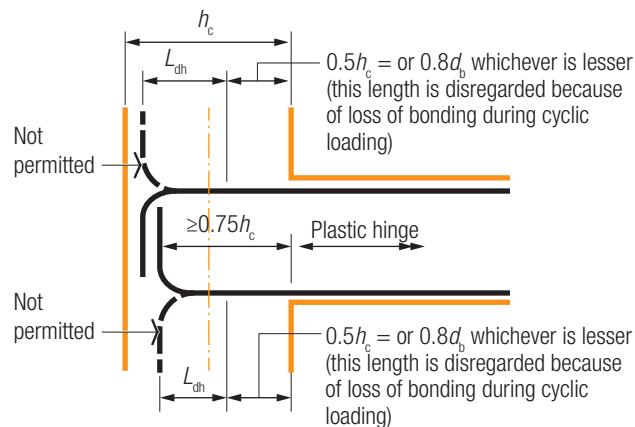
Beam column joints occur where the floor system or beams are connected to the columns.

Under seismic loading, the reversing moments induced above and below the joint in the column, and simultaneously occurring reversals of beam moment across the joint, cause the region to be subject to both horizontal and vertical shears of much greater magnitude than those experienced by the adjoining beams and columns themselves **Figure 46**. Even if the calculated shear force is low in a joint, transverse reinforcement should be provided through the joint to prevent the occurrence of brittle joint shear failure, rather than the desired flexural beam hinges⁵ **Figure 47**.

Where bending moments from the floor systems are transferred to the column, then lateral shear reinforcement is required in the beam column joint as specified in Clause 10.7.4.5 of AS 3600. This shear reinforcement is not required for an OMRF where the restraint of approximately the same depth is provided on all sides by the floor/beam system. All external



(a) Shear transfer of concrete compression forces and some bond forces from longitudinal bars by diagonal compression strut mechanism (from Goldsworthy³⁵)



(b) Anchorage of beam bars when the critical section of the plastic hinge forms at the column face (from NZS 3101 Part 2:1995)

Note

h_c = overall depth of column in the direction of the horizontal shear force, mm

Figure 46 – Shear transfer at beam column connections and recommended reinforcement anchorage at external column positions

columns, however, require this shear reinforcement as they are not fully restrained. The shear reinforcement is provided by continuing the closed fitments of the column into the beam column joint. However, good design practice requires that this shear reinforcement be provided at all beam column joints where the seismic design is a significant issue, even if not explicitly required.

The area of shear reinforcement required within the floor system/beam column joint is $A_{sv} \geq 0.35bs/f_{sy,f}$ and the spacing is:

- For an OMRF - s equals D_c or $15d_b$ for single bars and $0.5D_c$ or $7.5d_b$ for bundled bars (CI 10.7.4.3(b) of AS 3600)
- For an IMRF - maximum of $2s_c, 10d_b$ or 200mm. Refer to Figure 43 requirements from NZS 3101.1 and ACI 318M-14.

Column joint shear reinforcement unless restrained by beams on all four sides of approximately the same depth

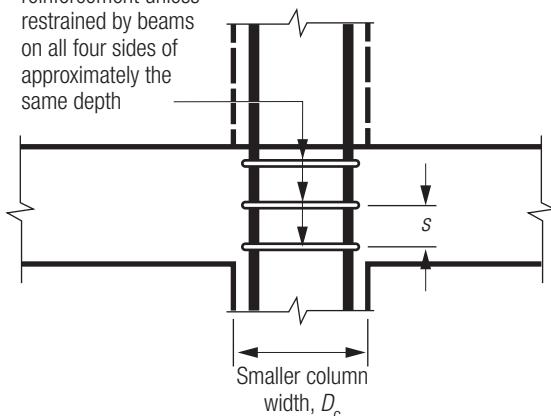


Figure 47 – Required shear reinforcement at floor slab/beam column joint for an OMRF
(From Reinforcement Detailing Handbook,Z6³³)

5.15 Precast and tilt-up concrete

5.15.1 General

Whilst load bearing precast and tilt-up structures have not had the same level of testing^{46 47 48 49}, experience has shown that precast and tilt-up elements, also known as prefabricated concrete elements in AS 3850.1 Prefabricated concrete elements - Part 1: General requirements, can be used successfully in seismic design if adequate attention is paid to connection details. Inadequate detailing has led to catastrophic failures in past earthquakes.

With precast and tilt-up structures, designers must create a rational structure from individual precast or tilt-up elements so that they are effectively tied together to resist seismic actions. Some precast and tilt-up elements may carry vertical actions, some may resist lateral actions, both in the plane of and perpendicular to the element, and some will have a combination of these actions. As with insitu concrete, failure of any vertical elements supporting loads under seismic actions can lead to collapse.

In New Zealand, there have been concerns about some horizontal precast floor systems. These concerns include the loss of support due to the insufficient allowance for construction tolerances, minimum bearing, and likely beam elongation, relative rotation between beam and precast unit and spalling of cover concrete under seismic actions. A minimum bearing of 100 mm, where practicable, should always be provided for horizontal precast units.

Other issues include incompatibility between the edge beam and the precast floor unit in the direction of the span, positive moment failure close to the face of the support (including dowel/plying action of the concrete plug cast into the ends of hollowcore units) and negative moment failure at the end or close to the end of starter bars in the topping concrete or reinforcement in filled webs if these are present **Figure 48**.

As for insitu concrete walls, designers must satisfy themselves that the calculated drift requirements can be resisted adequately without premature failure in the precast or tilt-up walls. Precast concrete walls may not have adequate bending capacity when subjected to large lateral displacements (drift) perpendicular to the face of the wall. As discussed for insitu walls, precast walls reinforced with a central layer of reinforcement, may be adequate for vertical actions, but may be inadequate for large lateral displacements both in-plane

and perpendicular to the face of the wall. Such walls often have poor ductility and may not survive a major earthquake. Walls with a central layer of reinforcement are not recommended for IL3 and IL4 structures due to their poor ductility at overload and under cycling loading.

The principal method to obtain structural integrity in precast and tilt-up structures, is the tying together of the precast/tilt-up elements in the transverse, longitudinal and vertical direction to form a complete structure. This will effectively interconnect all individual elements, provide stability to the structure, and provide redundant load paths.

Although tie systems are necessary for all concrete buildings, for precast and tilt-up concrete buildings there are more onerous requirements for loadbearing wall structures and the like. However, loadbearing wall structures tend to have large numbers of lateral resisting elements and may require only a limited number of ties. Framed structures, on the other hand, with relatively few lateral resisting elements will require careful placement of reinforcement over and above that required for direct structural actions.

5.15.2 Tie systems

All precast and tilt-up structures should have an effective tying system to provide robustness and to mobilise adequate structural integrity and redundancy within the structure as shown in **Figure 49**. Tie bars are placed within the floor in longitudinal, transverse and vertical directions to provide continuous tensile capacity throughout the structure. Their role is not only to transfer forces between the precast or tilt up units, from earthquake, wind and other actions, but also to give additional strength and robustness to the structure to withstand, to a certain extent, loading conditions termed as '*abnormal actions*'.

Tie requirements are determined either from the calculation of diaphragm action of the floor or by deemed-to-satisfy standard requirements. AS 3600 does not cover this design and detailing, but Eurocode 2⁵⁰ provides simple rules for the latter. Designers will need to make suitable allowance for different load factors and response factors.

Ties are in effect the minimum reinforcement requirements for a precast floor system. Chord and shear reinforcement required as part of the floor diaphragm can be included as part of the tie reinforcement and, in some cases, may exceed the deemed-to-satisfy requirements. Examples of tying together precast

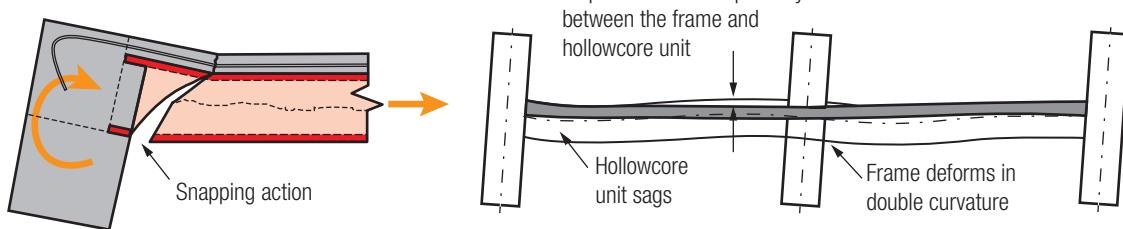
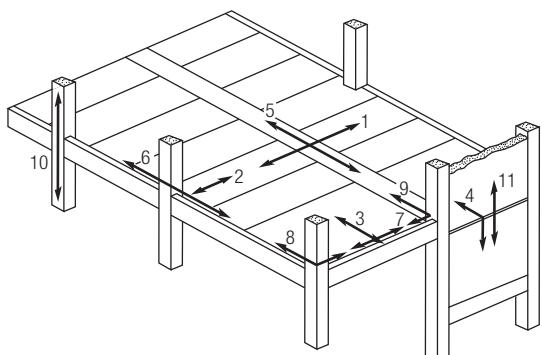


Figure 48 – Performance of precast flooring systems under earthquake actions

elements are shown in **Figure 49**.

Precast structures are more susceptible to the effect of abnormal actions than some traditional forms of construction because of the presence of joints between the structural elements. However, experience has shown that it is possible to manage these issues by effectively tying together the various elements of the structure and by correct detailing.

Designers should refer to the *Precast Concrete Handbook*⁵¹ and the NZ Guidelines for the use of Structural Precast Concrete in Buildings⁵².



- | | |
|-----------------------------|--|
| 1 Internal floor fitments | 7 Peripheral fitments |
| 2 Peripheral floor fitments | 8 Corner column fitments and peripheral fitments |
| 3 Gable internal fitments | 9 Column fitments |
| 4 Floor to wall fitments | 10 Vertical column fitments |
| 5 Internal beam fitments | 11 Vertical wall fitments |
| 6 Peripheral fitments | |

Figure 49 – Example of ties for precast systems for earthquake actions

5.15.3 Seismic design concepts for precast and tilt-up concrete

The main criteria to consider are:

- **Buildability** – the designing and detailing of structural elements such that they may be produced economically and erected easily and quickly while providing structural adequacy.
- **Ductility** – the level of ductility assumed in the design must be achieved in practice both by the precast elements and their connections. Where welded or bolted connections are used, detailing needs to be robust so that these do not fail under seismic actions. Section 8.2 of AS 1170.4 requires a component ductility factor, $R_c=1.0$ for such connections ie an increase of 2.5 times the design action being designed for.
- **Continuity** – continuity is used in precast flooring systems to resist diaphragm forces. Flexural continuity may be achieved by placing reinforcement in the topping concrete at the ends of the precast flooring units. However, the designer needs to exercise care to ensure that the topping thickness is sufficient to allow full anchorage and restraint of the reinforcement; an edge beam should preferably be provided.

- **Robustness** – the need for a structure to be able to maintain its overall integrity, without collapse of all or a significant part of the structure in the event of premature failure of a loadbearing element is well recognised. This need is especially important in precast construction where a number of discrete elements are connected to form the whole. The designer must therefore consider detailing as an essential part of the design in order to achieve a structure that will act in a monolithic fashion during a seismic event, eg P-delta effects due to plastic hinge formation, or inter-storey drift requiring consideration of minimum seating dimensions to prevent loss of support of precast floors or stair sections.

5.15.4 Precast slabs and structural toppings for diaphragm action

Where precast flooring elements are used, adequate reinforcement in the insitu structural topping (at least 50 mm thickness) has to be incorporated to provide suitable diaphragm action. The designer must ensure that the topping is adequately bonded to the precast elements if composite action is required in conjunction with adequate interface roughening. Without this, separation can occur, and the topping may buckle when subject to diagonal compression resulting from diaphragm shear, and be unable to transmit the floor inertial forces to the shear walls or columns. The structural topping must never be chased nor have slots cast into it at locations that could compromise the structural continuity of the topping.

Designers should ensure that not only is there an adequate load path for the forces that need to be transferred between a diaphragm and any lateral-force-resisting elements, such as walls or frames, but also that the connections are detailed such that they can adequately transfer the anticipated actions.

5.15.5 Tilt-up and precast buildings consisting principally of wall panels

These are a common form of construction for low-rise industrial and commercial buildings and medium-density residential buildings in Australia.

Designers must consider the diaphragm/wall connection design and detailing so that the connections can satisfactorily transfer the earthquake forces across the interfaces. Clause 5.2.2 of AS 1170.4 specifies that the structure needs to be tied together and sets out the design forces for buildings. These connection requirements are not usually onerous compared with, say, the typical detailing in New Zealand, due to the inherent rigidity of the structure. Precast and tilt-up buildings can be designed and detailed for limited ductility, provided sufficient attention is paid to the force transfer between panels and vertical and horizontal elements.

Stitch plates used to tie precast and tilt-up panels to form a single wall must be ductile, designed not to yield and should be at least 1.5 times the required capacity or as required by Section 8.2 of AS 1170.4. Stitch plates are not recommended where $\mu > 3$; in these situations, insitu stitch joints should be used. An example of a stitch plate is shown in Figure 50.

5.16 Footing and foundation systems

5.16.1 General

Suitable footings and foundations* are critical for ensuring that the structure will be able to resist both permanent and imposed actions and wind and seismic forces.

Foundation conditions vary widely in Australia and sometimes across the site. Where footings are founded on soft materials with a high water table, then consideration should be given to liquefaction. Although observed in early Australian earthquakes, liquefaction is thought only to be a possible problem in coastal areas with soft soils and shallow water tables.

A geotechnical investigation must be carried out for all buildings, including determination of the site sub-soil class that has a significant effect on the soils site amplification and design ground motion intensity. The design engineer must also consult with the geotechnical engineer on various aspects of the design of the footings for seismic actions.

From all these investigations and considerations, different footing options may be considered, and the footing system(s) chosen. Footings are usually one, or a combination, of the following:

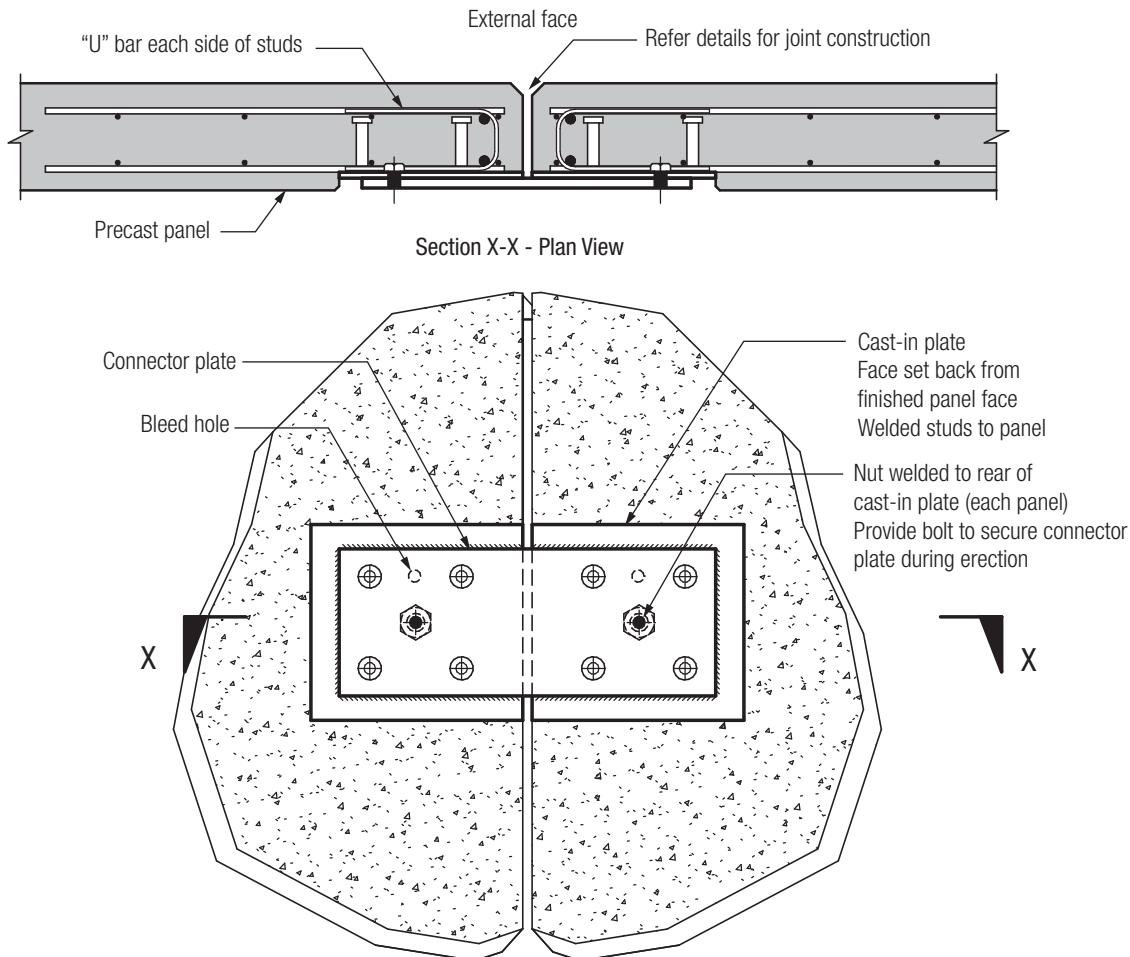


Figure 50 – Example of stitch plate

(Detail courtesy of Wallbridge & Gillbert)

- Pad (spread) footings or combined footings
- Strip footings
- Piled (or pier) footings
- Raft footings
- Balanced or coupled footings.

Where there are basements, the drift within the basement areas is likely to be small, and the foundation should be elastic under any conditions.

The designer should consider the effects of subsidence or differential settlement of the foundation material under the earthquake actions determined for the structure.

AS 1170.4 requires that footings supported on piles, or caissons, or spread footings that are located in or on soils with an ultimate bearing capacity of less than 250 kPa be restrained in any horizontal direction by tie beams or other means. This is to limit differential horizontal spread of the footings during an earthquake; a typical arrangement is shown in **Figure 51**. The floor slab can also be used if appropriately detailed. The minimum connection design action in Clause 5.4.2.1 of AS 1170.4 is 5% and therefore tie beams between footings should be designed for a minimum load in tension or compression equal to 5% of the ultimate column load.

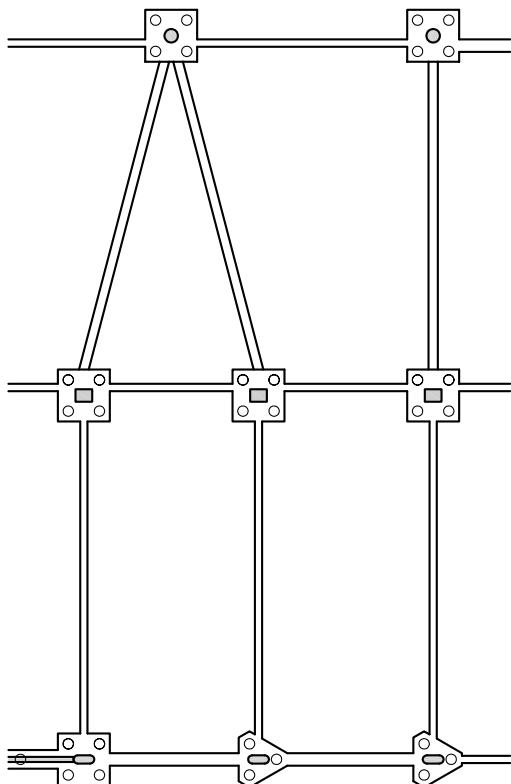


Figure 51 – Tie beams and pile caps

(Detail courtesy of Wallbridge & Gilbert)

Paulay and Priestly⁶ state that designers must clearly define how the earthquake-induced structural actions are transmitted through the proposed footing system to the foundations. They suggest a number of different types of footing systems and show how they respond to the seismic actions. Two of these will be relevant to Australian designers:

- **Elastic Footing Systems.** In regions of low seismicity, such as Australia, and for low-rise buildings with structural walls, it is possible to design and detail the entire structure to respond within elastic limits. However, if the annual probability of exceedance is 1 in 500 years, then the possibility of the design forces being significantly exceeded in a rare event should be considered.
- **Ductile Footing Systems.** In particular cases, the potential strength of the superstructure with respect to the specified seismic forces may be excessive (eg large shear wall structures). It may therefore be preferable for the footing system rather than the superstructure to be the principal source of energy dissipation during the inelastic response. A potential drawback of this system is that damage may occur during moderately strong earthquakes. Large cracks may form if yielding of reinforcement has occurred. Further, repairs to footings, if required, may be difficult or impossible and costly, particularly if below the water table or underneath a floor slab or similar.

Booth⁵ has an excellent discussion on the design of footings for seismic actions.

Footings for ductile cantilever shear walls must be capable of resisting the bending moment from the wall with a significant reserve of strength. The shear forces from the walls must also be transferred in the soils and for weak soils bending of the piles might be the only way to resist such shear forces.

5.16.2 Footing structures for moment-resisting frames

AS 3600 only provides some limited guidance on footing design and detailing for seismic actions. Additional precautions are warranted in difficult ground conditions.

AS 1170.4 appears to assume that there is no possibility for inelastic deformations to develop under earthquake actions and that standard detailing of footings for gravity, wind and seismic actions will be sufficient.

5.16.3 Isolated pad footings

These can present a problem with sliding, bearing capacity failure, or overturning if a plastic hinge forms at the base of the column. Unless precautions are taken, permanent deformation of the foundation can occur due to plastic deformation of the soil despite both the column and footing remaining elastic. If the column is assumed to be pinned at the base and the lateral forces are resisted by the cores or shear walls, they should be detailed to permit this pin connection to develop.

* For the purpose of this Guide, footings are the concrete elements of the building supported on the foundation which is the ground and may vary from soft soils to hard rock.

5.16.4 Combined footings and raft footing

Large moments transmitted by plastic hinges at column bases can be resolved by the use of stiff tie beams between pad footings or a raft footing, which provides a high degree of elastic restraint against column rotations. With this design approach, reinforcement yielding is unlikely to occur; there are therefore no special detailing requirements for ductility. However when utilising this strategy, designers would need to verify that the tie beams have sufficient reserve strength to accommodate the development of the column hinges.

To reduce the bearing pressure under footing pads, they may be joined to provide one continuous footing, creating a raft footing.

Stub columns require special consideration if inelastic deformations and shear failure are to be avoided. Paulay and Priestley⁶ consider that plastic hinges should, therefore, be restricted to the column section immediately above the beam or floor system.

5.16.5 Piled footings

Piled footing systems supporting structural walls and columns may be subject to large concentrated forces due to overturning moments and shear forces, and careful design is required. The junction between the pile and the pile caps is a highly stressed area where large curvatures can occur; adequate confinement and good connection details are vital. The end region of a pile under the pile cap must, therefore, be detailed to ensure full confinement of the longitudinal reinforcement using closed or helical ties which should extend into the pile cap. Most pile caps are designed using the strut and tie methods outlined in AS 3600.

Raking piles should be avoided as they can attract much of the dynamic actions from the superstructure and have been observed to fail in many severe earthquakes.

The locations of peak moments in the pile may necessitate the length of confinement being considerably extended. Even if calculations show no tension actions, minimum longitudinal reinforcement must be provided. The arrangement of longitudinal reinforcement should be as for columns, and the reinforcement should be fully anchored into the pile cap. In non-critical regions, nominal transverse ties or spiral hoops should be provided. Paulay and Priestley⁶ recommend that the vertical spacing does not exceed 16 times the diameter of the longitudinal bars. The shear design of piles needs careful consideration, particularly in the region immediately below the pile cap and is particularly important in soft soil conditions.

5.17 Stairs and ramps

Stairs and ramps are important structural elements, often ignored, and can be difficult to model in the design of buildings for earthquake actions.

The Christchurch earthquakes showed that these are an essential part of the building in the event of an earthquake. Assuming that the lifts no longer work the stairs are the primary means of egress from a building. There were many cases in Christchurch, where due to a lack of redundancy the stair flights failed and fell down the stair shaft, preventing the evacuation of the building Figure 52.

When detailing stairs, designers must carefully consider the inter-storey drift between adjacent floors. A recent amendment to AS 1170.4 requires detailing to accommodate 1.5 times the calculated inter-storey drift. Inadequate assessment of inter-storey drift may also result in the stiffness of the building being increased by the stairs or ramp. This inter-storey drift in turn may induce actions within the stair, causing the stair flights to collapse or lose support particularly if the stair flight is not supported within a robust shaft⁵³. Typical details of concrete stairs are shown in Figure 53.

Clause 5.2.3 of AS 1170.4 makes it clear that stiff components including stairs and ramps must be considered part of the seismic-force-resisting system or be separated such that no interaction can take place as the structure undergoes deflection due to earthquake effects, as determined by the Standard.



Figure 52 – Collapsed stairs to the Hotel Grand Chancellor

(Photograph courtesy of Dunning Thornton Consultants Ltd, NZ)

Where the stair is within a concrete shaft, often the stiffness of the stair is ignored. Such stairs failed badly in the Christchurch earthquake due to lateral and horizontal movements. Where the stair is within an area supported by columns then either the stair needs to be included in the analysis or the stair separated so that it can move independently under seismic actions.

Landings are often anchored to the wall using 'pullout bars' or cast-in ferrules, but these must be located beyond areas where the concrete may spall. Drilling and grouting of bars must not be used due to the risk inherent with post-construction installation, leading to poor and inadequate anchorage as a result of lack of bond or concrete spalling.

5.18 Fixings into concrete

Fixings into concrete can include cast-in bars, ferrules and bolts and the like and post-fixed anchors including chemical anchors or bars epoxied or grouted into the concrete at joints and similar.

In seismic events, cast-in bars and anchors such as bolts and ferrules must be sufficiently anchored away from the external face, so that should spalling of the concrete cover occur under seismic actions, they have adequate anchorage beyond the spalling.

Similar comments apply to post-fixed anchors. Also, many post-fixed anchors are not suitable for use in cracked concrete which occurs under seismic actions.

The use of reinforcing bars and anchors which have been '*glued*' in place may not be adequate under seismic conditions.

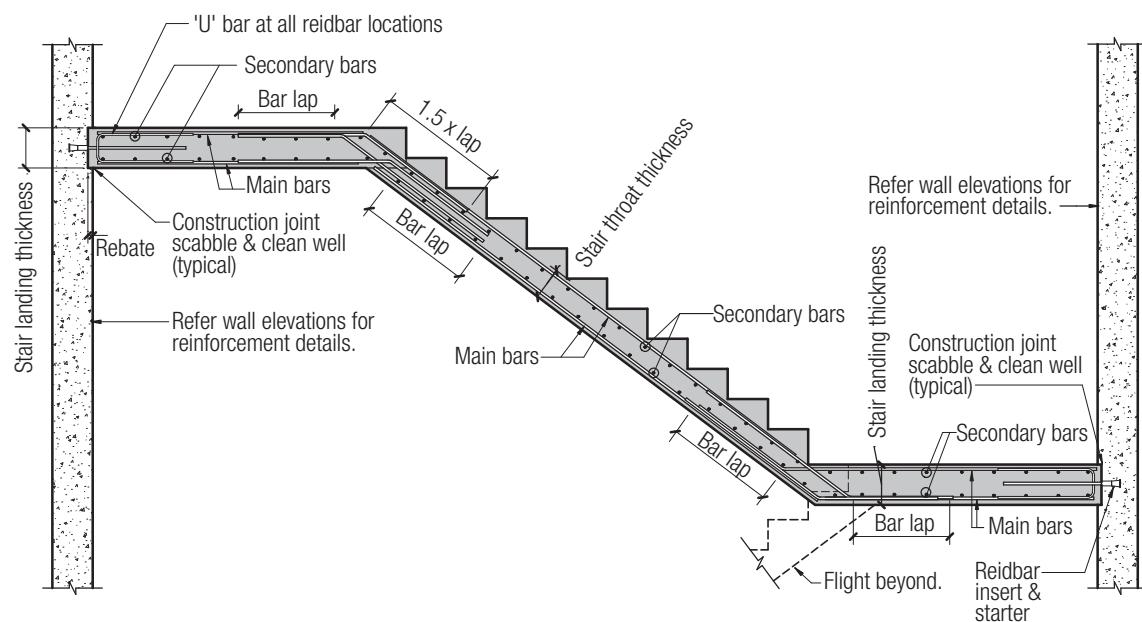


Figure 53 – Typical details for reinforced concrete stairs

(Detail courtesy of Wallbridge & Gillbert)

6 Existing buildings

While the Guide deals principally with new buildings, existing buildings is a complicated area, and designers need to be careful in assuming that an existing building is satisfactory for current seismic requirements.

AS 3826¹⁶ covers upgrading of buildings, but it is not called up in the BCA and refers to the previous Earthquake Standard.

For heritage structures, designers should consult with the relevant state heritage authority. Where an existing building has no additional floors constructed (and the seismic weight is similar) through consultation with the building certifier or building authority these buildings may be deemed compliant and no further design is required. However, where required or where additional levels are to be added, or the seismic load is changed, then the building will almost certainly need to comply with the current Standards and be strengthened.

Also, with existing buildings, particularly shopping centres and the like, that may have been constructed in various stages over time, it is critical to confirm that the structure matches the structural drawings. This may require intrusive site investigations and a dynamic analysis may provide a better overview of the performance than a static analysis because of the various configurations.

7 Documentation

7.1 Information to be shown on the drawings

Clause 1.4 of AS 3600 stipulates design details that must be shown in the documentation. These requirements are often ignored, but compliance with this clause is important. It is reproduced below.

1.4 DOCUMENTATION

The drawings and/or specification for concrete structures and members shall include, as appropriate, the following:

- (a) Reference number and date of issue of applicable design Standards.
- (b) Imposed actions (live loads) used in design.
- (c) The appropriate earthquake design category determined from AS 1170.4.
- (d) Any constraint on construction assumed in the design.
- (e) Exposure classification for durability.
- (f) Fire resistance level (FRL), if applicable.
- (g) Class and, where appropriate, grade designation of concrete.
- (h) Any required properties of the concrete.
- (i) The curing procedure.
- (j) Grade, Ductility Class and type of reinforcement and grade and type of tendons.
- (k) The size, quantity and location of all reinforcement, tendons and structural fixings and the cover to each.
- (l) The location and details of any splices, mechanical connections and welding of any reinforcement or tendon.
- (m) The maximum jacking force to be applied in each tendon and the order in which tendons are to be stressed.
- (n) The shape and size of each member.
- (o) The finish and method of control for unformed surfaces.
- (p) Class of formwork in accordance with AS 3610 for the surface finish specified.
- (q) The minimum period of time after placing of concrete before stripping of forms and removal of shores.
- (r) The location and details of planned construction and movement joints, and the method to be used for their protection.

Table 2 provides an example of the information required from Part (c) of Clause 1.4 of AS 3600

Table 2 - Example of Earthquake Design Category (EDC)

EARTHQUAKE LOADS	
HAZARD FACTOR:	Z=0.10
SITE SUBSOIL CLASS:	D _e
PROBABILITY FACTOR:	k _P =1.0
EARTHQUAKE DESIGN CATEGORY	II

7.2 Use of standard details

Standard details are developed in design offices based on the standard requirements to save documentation time and to improve design efficiency and quality control. However, designers must ensure that the standard details are appropriate for the structure being designed and that they comply with the current Standards. Too often, standard details are used without adequate review or consideration of their suitability for the structure to be built.

8 Non-structural components

While this Guide is about the design and detailing of concrete structures, designers cannot ignore non-structural components as they can significantly affect the use of a building after an earthquake. There is often inadequate separation of non-structural elements such as infill walls, windows, partitions, building services and precast concrete cladding to allow for the drift. Also, the failure of services such as fire or water can result in significant damage to the building.

Consideration of non-structural parts and components is particularly important in IL4 buildings with a post-disaster function. Such structures are required to remain operational after an earthquake, but may be prevented from doing so by failure of ceilings, partitions and services and the like.

In particular the failures of the ceilings may inhibit the evacuation of a building. Many failures of ceilings were observed in the Christchurch earthquake⁵⁴.

The legendary Olive View Hospital in California epitomises our understanding of earthquake design and how earthquake design has progressed over the last 50 years. The hospital was built in the late 1960s to the earthquake standards at that time, and was opened in December 1970. It was severely damaged, only three months after it was opened, in the San Fernando earthquake (9 February 1971) and was abandoned and demolished in 1973.

The hospital was then rebuilt some years later to stricter design and construction standards and the structure performed satisfactorily during the 1994 Northridge earthquake, but it was unusable because of the failure of building services and non-structural items. The ceilings and services were rebuilt to a higher standard, but to date there has been no major earthquake to test whether they have finally got it right. However should

the building performance still be inadequate in a major event then using the American baseball terminology, with three strikes it should be out.

A structural engineer will design and detail the building structure for earthquake actions, but usually not the non-structural components such as partitions, windows, access floors and ceilings. Similarly, mechanical and electrical components such as air-conditioning, fire and smoke detection systems, plant and equipment, air-conditioning ducts and piping, will normally be designed by others **Figure 54**.

The BCA requires all buildings to be designed to AS 1170.4 (assuming the deemed to comply methods of design are used) including all non-structural parts and components and their fastenings. Section 8 of AS 1170.4 requires that non-structural parts and components and their fastenings, as listed in Clause 8.1.4, be designed for horizontal and vertical earthquake forces as defined in Clauses 8.1.2 and 8.1.3.

Where non-structural parts and components have to be designed for wind and/or earthquake actions, the contractor, subcontractor or supplier will usually be responsible for their design. Contractors and subcontractors normally do not have the experience or design skills, so the supplier may have to provide this design advice. Many suppliers will struggle with the design requirements without assistance.

Where structural design is required for non-structural parts and components, the design should be carried out by a structural engineer experienced in the design of buildings for seismic actions in Australia using the Standards and design brief and specification together with a good understanding of how the parts and components are to be built and assembled.



Figure 54 – Services in a modern hospital corridor above the ceiling

(Photograph courtesy John Woodside)

9 Checklists

To help the client/building owner, designers, and the builder/contractor, the following checklists provide a series of questions which need to be answered when designing a reinforced concrete structure for earthquake actions. Note the checklists are not exhaustive, and designers need to be responsible in their use so as to not miss project specific issues that may be important for seismic design.

9.1 Client/building owner

- Do you understand that the design of the building for earthquake actions as set out in the Australian Standards is a life safety measure to try to ensure that the occupants can escape the building in the event of an earthquake, but that your building may be significantly damaged in a major earthquake?
- Do you understand that while earthquakes are low probability, significant damage is possible for your building in a major earthquake event unless a design approach is taken to minimise the damage? However, using an upgraded design will not necessarily prevent some damage in an extreme earthquake event.
- Is your business or operation critical in terms of continuing to operate after an earthquake or can you manage in new premises (if available) and cope with the disruption of setting them up?
- Do you have business interruption insurance for loss of business in the unlikely event of an earthquake?
- Do you store copies of your data and records off-site so that if in the event of an earthquake you are unable to access your building and can you continue to run your business or operation?
- Other design criteria to be discussed.

9.2 Principal Engineer responsible for analysis

- Do you have the qualifications and necessary design experience for the type of structure and the level of earthquake actions?
- Have you discussed with the client/owner and the design team the consequences of an earthquake and the design level you propose to adopt for the design of the building?
- Have you discussed the design of non-structural items required by other team members and subcontractors?
- Does the building have post-disaster function and what are the design criteria for that?
- Have you prepared a design criteria/design philosophy for the building and has it been checked independently to confirm that your approach is appropriate for the building to be designed, prior to starting the final design of the building?
- Have you submitted this design criteria/design philosophy for the building to the other team members and the owner, as a record of the agreement?
- Have you examined the individual structural elements to see if they can resist seismic and gravity load actions in an acceptable ductile manner?
- Have you considered using simple methods of analysis such as the equivalent static method in each orthogonal direction to identify load paths through the structure and the individual structural elements?

- From the load path assessment to identify the load paths through the different structural elements and zones where strains may be concentrated, have you determined where the load path depends on non-ductile material characteristics, such as the compressive strength of concrete?
- While the initial lateral strength of a building may be acceptable, critical non-ductile weak links in load paths may result in rapid degradation in strength during an earthquake. Have you identified these links and allowed for this degradation in assessing potential seismic performance? The ability of a building to deform in a ductile manner and sustain its lateral strength is more important than its initial lateral strength.
- Have you detailed the concrete in accordance with AS 3600 as required?
- Other consultant design items to be determined.

9.3 Principal Engineer/engineer responsible for detailing

- Have you provided seismic detailing as required in AS 3600 as a minimum for the chosen structural ductility factor?
- Have you considered the shear requirements of the columns and the requirements of the fitments to take the shear actions under earthquake actions, in addition to the restraint of longitudinal reinforcement?
- Have you provided the shear reinforcement in the floor slab or beam column joint as required by AS 3600?
- For earthquake design for structural ductility factors of $\mu > 2$ have you considered adopting an $f'_c \leq 50$ MPa for the columns and walls for ductility?
- Have you designed and detailed the beam column joint details and the connection of beams to structural walls and columns?
- Have you designed and detailed the diaphragm and the connection between floors/roof acting as diaphragms and lateral-force-resisting elements including walls and columns?
- Have you ensured the level of confinement of columns so that they have enough ductility to sustain the maximum inter-storey drifts that may be induced in a major earthquake?
- Have you avoided lapping bars in tension in high-stress zones and ensured that all laps and splices are adequately detailed on the drawings? Typically, bottom bars of beams and slabs are lapped at the points of support and top bars in the middle third of the beam and slabs.
- Have you used the same size fitments and varied the spacing to suit shear requirements (with a limited number of different spacings)? The spacing must not exceed the lesser of the values given in Clause 8.2.12.2 of AS 3600, but may be less for earthquake design.
- Have you reviewed the beam-beam and beam column junctions especially when the column and beam widths are the same, as the beam bars and column bars may clash, assuming the same cover is used for both elements?
- Have you avoided cogging bars into columns because of the congestion it will cause? Top bars can sometimes run into the slab. If caged bars are required have you considered drop-in splice bars.
- Have you provided a minimum of two bars top and bottom to support the fitments and provide continuity of reinforcement?

- Have you provided continuity in longitudinal reinforcement at the supports for the bottom reinforcement and in the middle of the beam for the top reinforcement? Note that the area of the bars should be of the order of 25% of the reinforcement in the other face (where possible) to allow for reversal and robustness.
- Have you ensured that compression reinforcement is adequately restrained by the fitments?
- For cantilevers, have you ensured that the top bars are anchored well back in an area of low stress?
- If beams are shown in a schedule on the drawings, have you then checked the schedule to ensure that all detailing fits within the shape to be built?
- Have you detailed additional bottom bars over columns/supports in flat slabs as insurance against collapse following a punching shear failure of the concrete floor plate?
- Have you provided elevations, sections and details of complicated or unusual beams on the drawings? These should be drawn at a suitable scale to ensure that the reinforcement can be placed and fixed as intended.
- Have you ensured that construction joints in slabs and beams are properly considered and specified?
- Other consultant design issues to be determined.

9.4 Builder/Contractor

- Do you and your construction staff understand the importance of the earthquake design of the building?
- Has the scheduler of reinforcement understood the detailing requirements required for earthquake design?
- Have you involved your steel fixer in discussing how the reinforcement is to be fixed, the order of fixing and any problems in completing the required details as set out by the engineer? If there are many similar details, should you mock up a small section to check all of the fixing requirements?
- Have you arranged for the structural engineer or their representatives to inspect the reinforcement prior to concreting?
- Have your subcontractors and suppliers complied with the BCA requirements for the design of parts and components for seismic actions to Section 8 of AS 1170.4?
- Have you requested third-party certification demonstrating compliance of the reinforcing steel supplied to the structural design specifications that reference AS/NZS 4671 and AS 3600?
- Other subcontractor design issues to be determined.

10 Conclusions

Australia is an area of lower seismicity, of low probability but high consequence seismic events in comparison to areas such as California, Japan, and New Zealand. The provisions for both the design and detailing of reinforced concrete structures in Australian in accordance with the BCA and referenced Standards reflect this in the design and detailing.

Many building structures in Australia will typically be designed and detailed in accordance with the main body of AS 3600 using the specific clauses for detailing in each section of the Standard; as a result, the detailing requirements are no more onerous than would normally be required. Combined with loose-bar detailing and with efficient fabrication procedures (as recommended in the CIA Reinforcement Detailing Handbook³³) together with some suggested additional consideration to provide the levels of ductility and continuity of reinforcement, this will allow the structure to meet the anticipated earthquake demands satisfactorily during a life safety event.

Indeed, with some additional design and detailing to Appendix C and further considerations, the building can meet higher levels of earthquake resistance and minimise the damage if required by your client.

It is important to provide a minimum level of ductility in both beams and columns framing into a joint and to ensure adequate confinement of column reinforcement in beam column joints. This principle applies regardless of the type of structural system employed.

Columns, shear walls or ordinary moment-resisting frame systems that are not deemed to require detailing to Appendix C, should still however be detailed to allow continuity and avoid buckling of bars in compression should stress reversal occur. Also for columns the load should be kept below the balance point for robustness reasons. For centrally reinforced walls with a single layer of reinforcement, care is needed to ensure that the walls can comply with the drift and also ductility requirements to avoid failure.

As discussed in this Guide, the ground motion intensity of an earthquake is difficult to predict with any accuracy. Should an earthquake occur of significantly greater magnitude than that which has been designed for (which is always a statistical possibility), brittle failure and collapse could occur.

With a limited amount of additional appropriately detailed fitments and continuity reinforcement, plastic hinges can be induced to form at a given load. However, yielding will be ductile (gradual), even if the design earthquake load is exceeded ie the hinge will act as a 'fuse' preventing the transfer of the larger forces.

The choice for the designer is clear. A fully elastic response by the structure is uneconomic and cannot be guaranteed while an inelastic response is permitted by the BCA and referenced Standards and can provide reliable seismic performance and safety. To achieve this, ductile performance must be guaranteed to prevent a catastrophic collapse and probable loss of life. This minimum level of ductility required can be readily achieved by careful detailing and reduction of the axial stresses in the columns and walls below the balance point on the interaction diagram.

Precast and tilt-up concrete construction requires additional care in detailing to ensure connection detailing is satisfactory and that floors are adequately supported and will act as diaphragms to correctly transfer horizontal forces. Further attention is required with mixed precast and insitu concrete construction to ensure monolithic behaviour.

Comparable overseas experience has shown that reinforced concrete, being insitu and precast/tilt-up, is an exceptionally simple, suitable and cost-effective solution for building structures for all seismic conditions; for areas of lower seismicity such as Australia, it is eminently suitable. Designers and specifiers can remain confident of reinforced concrete's ability to function and to meet the needs of today's construction industry.

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